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TIMBER DESIGN AND CONSTRUCTION

TIMBER DESIGN AND CONSTRUCTION

BY

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PREFACE

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THIS volume, as originally prepared by the senior author, was published in 1909 with the title of Structural Details and a sub-title referring to timber framing. It has now been fully revised and brought up to date by the junior author, and the title changed to conform more closely to the contents of the book.

Chapter I contains the digested results of extensive experimental investigations and research on fastenings used in framing, including bolts, nuts, washers, spikes, nails, screws, dowels, pins, keys, and metal straps and plates. Since the theoretic and practical values of tests depend largely upon the conditions under which the experiments were made, and the limitations of space preclude their complete description, the original sources of information are given.

Joints in framing are described in Chapter II. The principal types of tension joints are illustrated by working out designs of tabled fish-plate joints, plain fish-plate joints, tenon-bar splices, and shear-pin splices. Among other types of joints described are, lap and scarf joints, bearing joints, dovetail joints, mortise-and-tenon joints, step joints, angle blocks, and metal shoes. An attempt has been made to present the subject of joints and fastenings in a systematic manner, and to include practical information on timber construction.

Wooden beams and columns are covered in Chapter III. Examples are worked out for the design of combination beams, deepened beams, trussed beams, and columns. The use of beams, columns, beam hangers and anchorages, post caps, and angle braces are explained and illustrated.

Chapter IV, on wooden roof trusses, contains the design of a bay of a building, including rafters, purlins, and truss. The design of the truss includes all details as well as the main members. Results of tests on trusses, as well as examples of truss construction from practice, are also given.

Chapter V contains examples from practice of timber structures, among these being slow-burning mill construction, trestle construction, truss bridges, arch centering, and miscellaneous structures. One article is on dry rot.

Chapter VI covers the subjects of commercial grading, tests of clear timber, tests of commercial timber, factors influencing strength of timber, and unit designing stresses. Unit stresses as adopted by the American Railway Engineering Association in 1927, which are based on the most extensive tests available, especially those of the United States Forest Service, are given. Three formulas are given for allowable unit stresses on inclined planes.

A great deal of attention to details is given in the examples on the design of joints, beams and trusses, for it is believed that the importance of a careful study of every detail can only thus be properly emphasized. In practice it seems to be the exception rather than the rule to give the same attention to details of timber structures as to those of steel. In the interests of sound engineering practice it is essential that all connections and details have the same degree of security as the framed members.

In several articles the order of design is given in full, with a view of economizing the time of the student, and of promoting systematic habits in making the computations required; these objects being regarded as important elements in efficient education and practice. The designs of tension joints in Chapter II, of deepened beams, trussed beams, and columns in Chapter III, and of the roof truss in Chapter IV, have been carefully revised in accordance with the recent standard unit-stresses given in Chapter VI.

A subject embracing so many details of design and construction cannot be treated fully in a single volume of convenient size, since it is necessary to observe its application in several different cases to obtain an adequate idea of the value or use of any specific detail. The engineering periodicals and transactions contain a large amount of such illustrative material, and hence numerous selected references to them are made a prominent part of this work. Besides directing the student in his reading on the topics included, they encourage him to form the habit of consulting these great cyclopaedias of engineering practice.

In the first edition grateful acknowledgments were made to a number of consulting and designing engineers for photographs, drawings, and special information, and to several engineering periodicals for permission to reprint illustrations, as indicated in the text. To these are now added similar acknowledgments to the National Lumber Manufacturers Association for photographs; to the Engineering News-Record, the American Railway Engineering Association, and the American Society of Civil Engineers for permission to reproduce drawings; to the Forest Products Laboratory, the Southern Pine Association, and the West Coast Lumbermen's Association for special material; and to H. D. Dewell, R. L. Hankinson, C. R. Harding, J. A. Newlin, C. E. Paul, J. P. Quinlan, J. E. Wadsworth, and other engineers who have kindly furnished information.

OCTOBER 12, 1929.

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TIMBER DESIGN AND CONSTRUCTION

CHAPTER I

FASTENINGS USED IN FRAMING

ART. 1. BOLTS AND NUTS

A BOLT consists of a rod with suitable attachments at its ends which is used to hold together two or more pieces of any structure. The kind of bolt most frequently employed is shown in Fig. 1*a*, and has a permanent head at one end, and has the other end threaded to receive a nut. It is called a screw-bolt when it is necessary to distinguish it from other kinds of bolts.

Bolts are classified according to the shape of the head, or some structural feature of the head, by the mode in which they are secured or held in place, or by the purpose for which they are employed. The terms used to designate the numerous kinds of bolts may be found by reference to the word "bolt" in the large dictionaries.

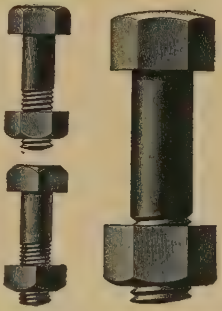
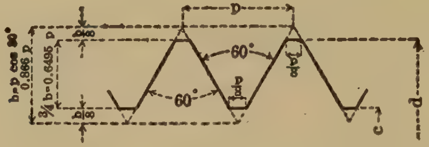


FIG. 1*a*.—Bolts.

Bolts are manufactured either as rough or finished. The former are commonly used for woodwork and for structural steel fitting-up work. Finished bolts may be either cut or turned. Cut bolts are made by pushing a rough bolt into a series of revolving cutters set to the desired diameter. Turned bolts are those turned to a desired diameter in a lathe. Turned bolts give more nearly exact dimensions than cut bolts. Cut and turned bolts are used where a tight fit is required, the holes being either drilled or punched and reamed to exact size. A common use is in bolting metal parts, such as in machinery and in structural steel work where the connections are bolted instead of riveted. The cost of rough, cut and turned bolts is in the approximate ratio of 3 to 7 to 10.

The standard forms and dimensions of screw threads in general use in this country were devised by William Sellers and recommended by the Franklin Institute in December, 1864 (Journal of the Franklin Institute, third series, vol. 47, page 344, and vol. 49, page 53). Table 1*a*, taken from the Carnegie Pocket Companion, gives complete screw-thread data.

TABLE 1*A*.



Diameter		Area		Number of Threads per Inch
Total *d*, In.	Net, *c*, In.	Total Dia., *d*, Sq. In.	Net Dia., *c*, Sq. In.	
¼	.185	.049	.027	20
⅜	.294	.110	.068	16
½	.400	.196	.126	13
⅝	.507	.307	.202	11
¾	.620	.442	.302	10
⅞	.731	.601	.419	9
1	.838	.785	.551	8
1⅛	.939	.994	.693	7
1¼	1.064	1.227	.890	7
1⅜	1.158	1.485	1.054	6
1½	1.283	1.767	1.294	6
1⅝	1.389	2.074	1.515	5½
1¾	1.490	2.405	1.744	5
1⅞	1.615	2.761	2.049	5
2	1.711	3.142	2.300	4½
2⅛	1.836	3.547	2.649	4½
2¼	1.961	3.976	3.021	4½
2⅜	2.086	4.430	3.419	4½
2½	2.175	4.909	3.716	4
2⅝	2.300	5.412	4.156	4
2¾	2.425	5.940	4.619	4
2⅞	2.550	6.492	5.108	4
3	2.629	7.069	5.428	3½
3¼	2.879	8.296	6.509	3½
3½	3.100	9.621	7.549	3¼
3¾	3.317	11.045	8.641	3
4	3.567	12.566	9.993	3
4¼	3.798	14.186	11.330	2⅞
4½	4.028	15.904	12.741	2¾
4¾	4.255	17.721	14.221	2⅝
5	4.480	19.635	15.766	2½
5¼	4.730	21.648	17.574	2½
5½	4.953	23.758	19.268	2⅜
5¾	5.203	25.967	21.262	2⅜
6	5.423	28.274	23.095	2¼

The proportions recommended by the Franklin Institute for nuts and bolt heads, however, have not been generally adopted because of the odd size of bar required to make them. The proportions in common use are known as the manufacturers' standard, whereas the Franklin Institute standard for screw threads, nuts and bolt heads is commonly known as the United States standard.

The thickness of a rough bolt head is made sufficient to develop the safe tensile strength of the bolt, but the nut is made thicker, or equal to the diameter of the bolt, probably to provide extra wearing surface for the thread. The finished head and nut are made of the same thickness to improve the appearance of the finished work, as they frequently alternate in position or come into close proximity.

In the Franklin Institute standard sizes, rough nuts and heads have a short diameter, expressed in inches, equal to $1.5d + \frac{1}{8}$, where d is the diameter of the bolt, whereas finished nuts and heads are $\frac{1}{16}$ in. smaller. Rough nuts have a thickness equal to d, while finished nuts have a thickness $\frac{1}{16}$ in. less. Rough heads have a thickness equal to one-half the short diameter of the nut, and finished heads have a thickness $\frac{1}{16}$ in. less. The short diameter is the diameter of the inscribed circle. This applies to both hexagonal and square nuts and heads. Table 1*b* gives values for the short diameter, d, and the thickness, t, for bolt heads and nuts on the basis of the manufacturers' standard. It will be noted that for some sizes there is more than one standard nut.

Under certain circumstances the thickness of bolt heads and nuts may vary from the standards. For instance, a rough bolt head for soft timber is often made large and thin, whereas a clamping nut, which is frequently screwed tight and unscrewed again, is made much thicker than the rule indicates for permanent fastenings.

With respect to the principal stresses in the body of the bolt, a bolt may be used either to prevent the separation of two or more parts of a structure in the direction of the bolts, or to prevent them from sliding past each other. In the former case the body of the bolt is subject to tension and in the latter case to compression and shear, either with or without flexure. When a bolt connects timbers which tend to slide on the surface of contact, the shear in any cross-section need not be considered, as the bolt will always bend more or less on account of the yielding of the wooden fibers near the plane of contact. In this case the hole should be bored so as to secure a close or driving fit. For this purpose, in bolting the longitudinal and sway braces to the bents of wooden trestle bridges it is customary to use an auger $\frac{1}{8}$ in. smaller in diameter than the bolt for bolts having a diameter of $\frac{3}{4}$ in. or over, and $\frac{1}{16}$ in. smaller for bolts with a diameter of $\frac{5}{8}$ in. or less. For the description and illustration of an ingenious

TABLE 1*b*.—DIMENSIONS OF BOLT HEADS AND NUTS

Manufacturers' Standard Sizes

Diameter of bolt	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{7}{8}$	2
Bolt head, *d*	$\frac{3}{8}$	$\frac{15}{32}$	$\frac{9}{16}$	$\frac{21}{32}$	$\frac{3}{4}$	$\frac{27}{32}$	$\frac{15}{16}$	$1\frac{1}{8}$	$1\frac{5}{16}$	$1\frac{1}{2}$	$1\frac{11}{16}$	$1\frac{7}{8}$	$2\frac{1}{16}$	$2\frac{1}{4}$	$2\frac{7}{16}$	$2\frac{5}{8}$	$2\frac{13}{16}$	3
Bolt head, *t*	$\frac{3}{16}$	$\frac{15}{64}$	$\frac{9}{32}$	$\frac{21}{64}$	$\frac{3}{8}$	$\frac{27}{64}$	$\frac{15}{32}$	$\frac{9}{16}$	$\frac{21}{32}$	$\frac{3}{4}$	$\frac{27}{32}$	$\frac{15}{16}$	$1\frac{1}{32}$	$1\frac{1}{8}$	$1\frac{7}{32}$	$1\frac{5}{16}$	$1\frac{13}{32}$	$1\frac{1}{2}$
Nuts, *d*	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	$\frac{7}{8}$	$1\frac{1}{8}$	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$	2*	$2\frac{1}{4}$*	$2\frac{1}{2}$*	$2\frac{3}{4}$*	3*	$3\frac{1}{4}$*	$3\frac{1}{2}$*	$3\frac{1}{2}$*
Nuts, *t*	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{3}{4}$*	$1\frac{3}{8}$*	$1\frac{1}{2}$*	$1\frac{5}{8}$*	$1\frac{3}{4}$*	$1\frac{7}{8}$*	2*	2*
Nuts, *d*	....	....	....	....	1	....	$1\frac{1}{8}$*	$1\frac{3}{8}$	$1\frac{1}{2}$*	$1\frac{3}{4}$*	2†	$2\frac{1}{4}$†	$2\frac{3}{4}$†	3†	$3\frac{1}{4}$†	$3\frac{1}{2}$†	$3\frac{3}{4}$†	$3\frac{1}{2}$*
Nuts, *t*	....	....	....	....	$\frac{1}{2}$	....	$\frac{3}{4}$*	$\frac{3}{4}$	1*	$1\frac{1}{8}$*	$1\frac{1}{8}$†	$1\frac{1}{4}$†	$1\frac{3}{8}$	$1\frac{1}{2}$†	$1\frac{5}{8}$	$1\frac{3}{4}$†	$1\frac{7}{8}$†	$2\frac{1}{8}$*
Nuts, *d*	....	....	....	....	1*	....	$1\frac{1}{4}$	$1\frac{3}{8}$*	$1\frac{5}{8}$	2†	$2\frac{1}{4}$†	$2\frac{1}{2}$†	....	....	....	....	....	4†
Nuts, *t*	....	....	....	....	$\frac{9}{16}$*	....	$\frac{5}{8}$	$\frac{7}{8}$*	$\frac{7}{8}$	1†	$1\frac{1}{8}$†	$1\frac{1}{4}$†	....	....	....	....	....	2†
Nuts, *d*	....	....	....	....	....	....	$1\frac{1}{4}$*	$1\frac{1}{2}$	$1\frac{5}{8}$*	....	....	....	....	....	....	....	....	....
Nuts, *t*	....	....	....	....	....	....	$\frac{3}{4}$*	$\frac{3}{4}$	1*	....	....	....	....	....	....	....	....	....
Nuts, *d*	....	....	....	....	....	....	....	$1\frac{1}{2}$*	$1\frac{3}{4}$†	....	....	....	....	....	....	....	....	....
Nuts, *t*	....	....	....	....	....	....	....	$\frac{7}{8}$*	$\frac{7}{8}$†	....	....	....	....	....	....	....	....	....

All dimensions in inches, *d* equals short diameter, *t* equals thickness.

* For hexagonal nuts only.

† For square nuts only.

device designed to increase the resistance of a bolted joint to shear in the plane of contact see an article on The Use of a Double-cone Washer for Timber Joints, by L. S. Austin, in Engineering News, vol. 52, page 348, Oct. 20, 1904.

The length of a bolt is the distance from the inside of the head to the end of the screw. The grip of a bolt is the distance between the inside faces of the washers, or the total thickness of the material to be held together.

The design of a bolt subject to tension only requires the computation of the net section area at the root of the thread and the selection of a diameter of bolt from a table of commercial sizes which provides a net area equal to or slightly exceeding the one required. If a bolt has to be screwed up when the maximum direct tension is acting, it is necessary to increase the net section to provide for the additional stresses due to torsion. This increase may be taken at approximately 15 percent.

Experiments made by W. R. King show that for a 1½-in. wrought-iron bolt 12 and 18 threads to the inch give respectively 21 and 23 percent greater strength than the standard number of 6 threads, and a corresponding increase of elongation of 140 and 220 percent. These results indicate that for some special purposes where a large number of bolts are required, it may be desirable to increase the number of threads per inch, probably not to exceed double the standard number. See a paper by Charles T. Porter in Transactions American Society of Mechanical Engineers, 1903, vol. 24, page 137.

In long bolts there is considerable waste of metal owing to the excess of section of the body of the bolt over that at the root of the threads at its ends. This is avoided by using what are technically known as upset screw ends. Their diameter is enlarged so that their strength is at least equal to that of the body of the bolt. To secure this result the area at the root of the thread must be somewhat larger than that of the main rod, since the unit tensile strength of the material is reduced by the process of upsetting. Rods having a diameter of 1 in. should be upset for lengths of 12 ft. and over, while 2½-in. rods should be upset for lengths of 6 ft. and over. For intermediate sizes the lengths may be varied between the limits given. For shorter lengths it is more economical to use rods without upset ends.

Tables of upset screw ends for both round and square bars are given in the manufacturers' handbooks, their proportions based on numerous tests of finished bars. To reduce the number of forms or dies required, the diameter of the upset varies by eighths of an inch, while that of the rods varies by sixteenths of an inch. For some data and discussion on the strength of upset and plain screw ends see Engineering Record, 1895, vol. 32, pages 83, 102, 120 and 257, and Railroad Age Gazette, vol. 45, page 611, July 31, 1908.

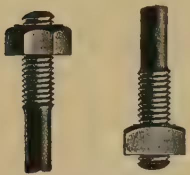


Fig. 1*b*.—Upset Screw Ends.

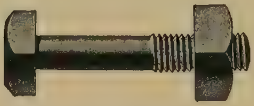


Fig. 1*c*.—Bolt with Cold-pressed Threads.

Instead of cutting the threads in a lathe, a process of cold rolling has been invented in which the threads and shoulders are made by depressing and shaping the fibers so as to retain their tensile strength. In another process the thread is raised slightly above the surface of the shank or body of the bolt by cold pressure, thus preserving the full strength of the body in the threaded portion (Fig. 1*c*). Some bolts are made with a helicoid shank which prevents the bolt from turning when the nut is screwed in place.

When there is danger of a nut's working loose under the influence of vibration or other causes, various devices are provided to lock the nut. One of these consists in driving a nail alongside into the timber through a slot in the washer (see Fig. 2*c*) provided for this purpose. A check or jam nut may be placed on top of the standard nut and brought into a firm bearing with it. Check nuts have the same diameter, but only a trifle more than half of the thickness, of the standard form. Still another device consists of a self-locking nut having a split side and tapped slightly smaller than the standard, so that when wrenched on the bolt the side opens a trifle, but the bolt is held with sufficient grip to prevent working loose. In case the nut is not intended to be unscrewed at any time after the bolt is put in place, it may be

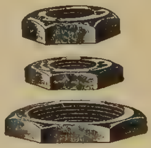


Fig. 1*d*.—Check Nuts.

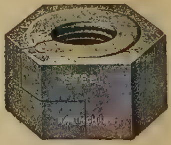


Fig. 1*e*.—Elastic Nut.

locked by deforming the adjacent outer threads with a center punch.

Problem 1. Find the diameter of a bolt required to resist 18,500 lb. in tension, the working unit stress being specified as 16,000 lb. per sq. in.

ART. 2. WASHERS

A washer is a perforated disk of iron or steel placed under the nut or head of a bolt. When the bolt passes through timber, the object of the washer is usually to provide sufficient bearing area so that the compression on the surface of the timber may not exceed a safe value. Sometimes a small washer may be used merely to avoid defacing the surface.

There are six types of washers used in timber construction: (1) cast-iron ogee washers, (2) cast-iron ribbed washers, (3) malleable-iron washers, (4) circular pressed-steel washers, (5) square steel-plate washers and (6) beveled cast-iron washers.

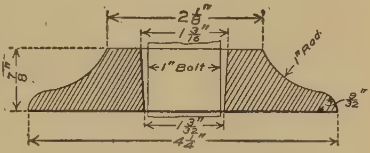


Fig. 2*a*.—Section of Ogee Washer.

The ogee washer is round with a cross section like that shown in Fig. 2*a*. On drawings it is often designated as "O. G. washer." For the Pittsburgh standards given in Table 2*a*, the size of the hole at the top or face is $\frac{1}{8}$ in. larger than at the back for the smaller sizes, and $\frac{1}{4}$ in. for the larger sizes. The diameter of the top or face of the washer is $(2d + 0.5)$ in., d being the diameter of the bolt.

Table 2*a*.—Cast-Iron Ogee Washers

Diam. of Bolt, Inches	Diam. of Washer, Inches	Thickness, Inches	Weight of 100, Pounds	Bearing Area, Sq. In.	Diam. of Bolt, Inches	Diam. of Washer, Inches	Thickness, Inches	Weight of 100, Pounds	Bearing Area, Sq. In.
$\frac{3}{8}$	$1\frac{1}{2}$	$\frac{5}{16}$	8.5	1.57	1	4	$\frac{7}{8}$	180	11.57
$\frac{1}{2}$	2	$\frac{3}{8}$	22	2.83	1	4	1	194	11.57
$\frac{1}{2}$	$2\frac{3}{8}$	$\frac{1}{2}$	37.5	4.12	$1\frac{1}{8}$	$4\frac{1}{2}$	1	215	14.68
$\frac{5}{8}$	$2\frac{1}{2}$	$\frac{1}{2}$	45	4.47	$1\frac{1}{8}$	$4\frac{1}{2}$	$1\frac{1}{8}$	295	14.68
$\frac{5}{8}$	3	$\frac{5}{8}$	75	6.63	$1\frac{1}{4}$	5	$1\frac{1}{8}$	320	18.15
$\frac{3}{4}$	3	$\frac{5}{8}$	72	6.47	$1\frac{1}{4}$	$5\frac{3}{8}$	$1\frac{1}{4}$	469	21.21
$\frac{3}{4}$	$3\frac{1}{4}$	$\frac{3}{4}$	100	7.69	$1\frac{3}{8}$	$5\frac{1}{2}$	$1\frac{1}{4}$	425	21.99
$\frac{7}{8}$	$3\frac{1}{2}$	$\frac{3}{4}$	115	8.84	$1\frac{1}{2}$	6	$1\frac{3}{8}$	525	26.20
$\frac{7}{8}$	$3\frac{5}{8}$	$\frac{7}{8}$	150	9.54	$1\frac{1}{2}$	6	$1\frac{1}{2}$	688	26.20

The proportions of ogee washers adopted by some of the manufacturers differ somewhat in details. For example, the diameter of the face is sometimes made $(2d + 0.125)$ in. Referring to Fig. 2*b*, the radius F for a $\frac{3}{4}$-in. bolt is $\frac{1}{2}$ inch in one case and $\frac{3}{4}$ in. in another, whereas the corresponding values of the radius G are $\frac{5}{16}$ and $\frac{7}{32}$ inch. In the form illustrated in Fig. 2*a* the radius of the upper curve equals the diameter of the bolt, whereas that of the lower curve is approximately one-fourth as large.

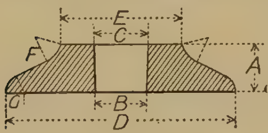


Fig. 2*b*.—Section of Standard Ogee Washer.

Most of the ogee washers supplied by the trade have an outside diameter approximately equal to four times the diameter of the bolt, although the bearing area when used on soft wood should be larger than when used on hard wood. Some of the soft woods like white and Norway pine require two or more times as large a bearing area under washers as hard woods like white oak. The thickness is made either equal to the diameter of the bolt or $\frac{1}{8}$ in. less.

Figure 2*c* shows a slotted washer, standard with the American Bridge Company, which differs from the ordinary form by the addition of a slot, through which a nail may be driven for the purpose of locking the nut in its final position. The dimensions in inches are given in Table 2*b*.

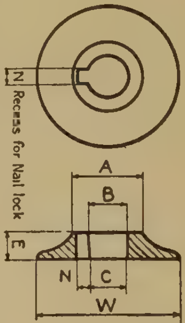


Fig. 2*c*.—Standard Ogee Washer, American Bridge Company.

Table 2*b*.—Standard Ogee Washers

American Bridge Company

Diam. of Bolt	*W*	*E*	*A*	*B*	*C*	*N*	Bearing Area, Sq. In.	Weight, Lb.
$\frac{1}{2}$	$2\frac{1}{4}$	$\frac{3}{8}$	$1\frac{1}{8}$	$\frac{5}{8}$	$\frac{9}{16}$	$\frac{3}{16}$	3.73	0.3
$\frac{5}{8}$	$2\frac{3}{4}$	$\frac{1}{2}$	$1\frac{3}{8}$	$\frac{3}{4}$	$\frac{11}{16}$	$\frac{5}{16}$	5.57	0.4
$\frac{3}{4}$	$3\frac{1}{4}$	$\frac{5}{8}$	$1\frac{5}{8}$	$\frac{7}{8}$	$\frac{13}{16}$	$\frac{5}{16}$	7.78	0.7
$\frac{7}{8}$	$3\frac{3}{4}$	$\frac{3}{4}$	$1\frac{7}{8}$	$1\frac{1}{16}$	$\frac{31}{32}$	$\frac{5}{16}$	10.31	1.0

Cast-iron washers of the ribbed type are designed to save metal over the ogee type. The washer consists of a base plate, spool and ribs; and sometimes a rim. Table 2*c* and Fig. 2*d* give the dimensions of cast-iron ribbed washers used at the Panama–Pacific Exposition, where the bearing was across the fiber of Douglas fir. It will be noted that these washers are much larger for a given size of bolt than the ogee washers shown above.

Table 2*c*.—Cast-Iron Washers, Panama-Pacific Exposition

Size of Bolt	Size of Upset	*a*	*b*	*c*	*d*	*h*	*t*	Type	Bearing Area, Sq. In.	Weight, Lb.
5/8	Not upset	3/4	1 9/16	1/8	3 1/8	3/4	1/8	I	7.23	0.56
3/4	" "	7/8	1 3/4	3/16	4	1	1/4	I	11.97	1.10
7/8	" "	1	2 1/4	3/16	4 1/2	1 1/8	1/4	I	15.12	1.80
3/4	1	1 1/8	2 3/8	1/4	5 1/4	1 1/4	1/4	I	20.66	2.79
13/16	1 1/8	1 1/4	2 5/8	1/4	5 5/8	1 5/16	1/4	I	23.62	3.29
7/8	1 1/4	1 3/8	3	5/16	6 1/2	1 7/16	5/16	II	31.70	5.30
1	1 3/8	1 1/2	3 1/4	5/16	7	1 9/16	5/16	II	36.71	6.34
1 1/8	1 1/2	1 5/8	3 1/2	3/8	7 7/8	1 11/16	3/8	II	46.64	9.04
1 1/4	1 5/8	1 3/4	3 3/4	3/8	8 5/8	1 15/16	3/8	II	56.02	11.30
1 3/8	1 3/4	1 7/8	4	3/8	9 1/8	2 1/4	3/8	II	62.64	13.59
1 7/16	1 7/8	2	4 3/8	7/16	10	2 3/8	7/16	III	75.40	18.66
1 1/2	2	2 1/8	4 5/8	7/16	10 3/8	2 7/16	7/16	III	81.00	20.39
1 5/8	2 1/8	2 1/4	4 3/4	1/2	11 1/4	2 5/8	1/2	III	95.42	25.99
1 3/4	2 1/4	2 3/8	5 1/8	1/2	12 1/8	2 3/4	1/2	III	110.97	30.62
1 7/8	2 3/8	2 1/2	5 1/8	5/8	11 1/4	4 1/4	5/8	IV	121.59	48.23
2	2 1/2	2 5/8	5 5/8	3/4	12	5 1/8	3/4	IV	138.59	69.32

Note: Provide 1/8-in. fillets on all washers.

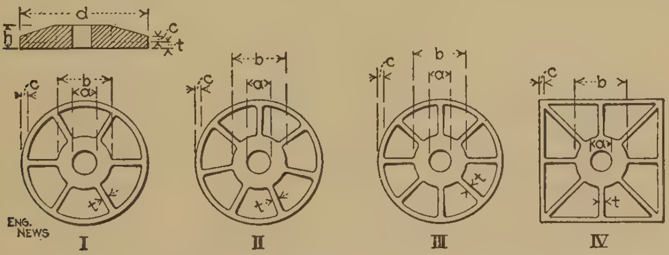


Fig. 2*d*.—Cast-iron Washers, Panama–Pacific Exposition.

They were designed for a bearing pressure on the timber of 350 lb. per sq. in., with a unit stress in the bolt of 16,000 lb. per sq. in.

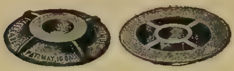


Fig. 2e.—Malleable Iron Washer.

Fig. 2f.—Circular Washer.

Fig. 2g.—Square Washer.

In malleable-iron washers, illustrated in Fig. 2e, the bearing area is reduced as the bottom plate does not extend to the spool or central cylinder.

The ordinary forms of steel washers are comparatively thin, the diameter ranging from two to about three times the diameter of the bolt. The circular ones are not intended to distribute on the wood the full tensile strength of the bolt, but the square washers are materially stronger, some sizes being proportioned for effective use on wood though others are not.

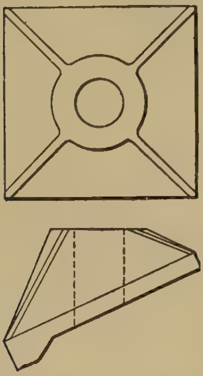
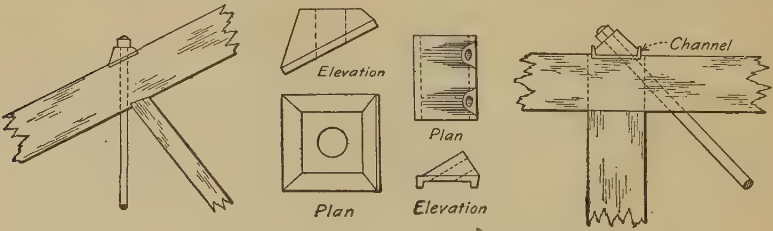


Fig. 2h.—Beveled Washer.

When the direction of a bolt is not perpendicular to the surface of the timber, a beveled washer is required unless only a slight notch is needed to secure a square bearing. Figures 2 *i-l* show three forms, one a solid casting, the next a double washer for a more acute angle between the bolt and timber surface, and the third a combination of two or more beveled washers with a channel, in order to distribute the



Figs. 2*i*, *j*, *k*, and *l*.—Beveled Washers.

bearing surface over the full width of the timbers. Still another form is illustrated in Fig. 2*h*.

In designing a beveled washer, the bearing surfaces on the end and base must provide the proper safe resistance to the respective components of the stress in the bolt, and the resultants of these three stresses must intersect in a point if it is desired to secure a uniform distribution of the compression on the two surfaces of the timber, as illustrated on Fig. 2*m*.

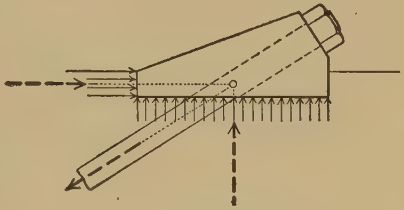


FIG. 2*m*.

A few other forms of washers are used for special purposes in construction. One is the cup washer used on guard timbers of railroad tracks on bridges and elevated railroads. It is sunk into the timber so as to prevent the bolt head from projecting above its surface. A similar form called socket washers is used in the floors of freight cars, elevators, warehouses, piers, and wharves for the same purpose. See also the reference to a double-cone washer in Art. 1.

When the bearing surface of a washer is inclined to the fibers of the wood, the relation between the allowable bearing pressure per square inch and that on the sides and ends of the fibers respectively may be obtained by the method given in Art. 91.

PROBLEM 2. A vertical bolt which is carrying a tensile stress of 8150 lb. passes through a timber inclined an angle of $19\frac{1}{2}$ degrees with the horizontal. The timber is $3\frac{3}{4}$ in. wide and the diameter of the upset end of the bolt is $1\frac{1}{8}$ in. The allowable unit stresses on the sides and ends of the fibers are 2000 and 700 lb. per sq. in. respectively. Design the washer required, and indicate its dimensions on a pencil drawing made to scale.

ART. 3. THE STRENGTH OF WASHERS

The standard commercial sizes of washers are supposed to resist with safety the allowable stresses in the corresponding bolts. For example, a $\frac{3}{4}$-in. bolt has an area of 0.302 sq. in. at the root of the thread, and for an allowable tensile stress of 16,000 lb. per sq. in., it can resist a stress of 4830 lb. The diameter of the corresponding washer in Table 2*a* is $3\frac{1}{4}$ in., and its net bearing area is

8.296–0.601 = 7.695 sq. in. The unit compression on the wood must therefore equal 4830/7.695 = 628 lb. This value exceeds the average elastic limit for compression on the side of the fiber for most of the species of wood used in construction (Art. 87). If, therefore, a washer and a bolt of the size given are used together in structural timbers of these species, either the bolt cannot develop its proper strength, or the wood is subjected to a greater compression than should be allowed in properly designed structures. In either case there is a theoretic lack of economy, even if no danger is directly involved. Experienced inspectors claim that, in general, evidences of weakness in timber structures, especially those exposed to the weather, are shown first in bearings on the sides of the fibers.

A manufacturers' standard square steel washer for a $\frac{3}{4}$-in. bolt is $2\frac{1}{2}$ in. square, $\frac{1}{4}$ in. thick, and has a hole 27/32 in. in diameter. Its net area is 5.691 sq. in. and hence the compression required to develop the working strength of the bolt is 4830/5.691 = 849 lb. per sq. in.

Tests made by F. S. Bixby and reported by H. D. Dewell[1] showed that standard cast-iron ogee washers developed the elastic limit strengths of Douglas fir in side compression at loads of from 32 to 40 percent of the elastic limit strengths of the bolts, whereas with washers of special design these values were increased to from 86 to 91 percent. The test results are as follows:

Diam. of Bolt, In.	Ultimate Strength of Bolt, Lb.	Standard Washer			Special Washer				
		Diam. In.	Area, Sq. In.	Load at Elastic Limit of Wood, Lb.	Diam. In.	Area, Sq. In.	Thickness, In.	Load at Elastic Limit of Wood, Lb.	Strength of Washer, Lb.
$\frac{1}{2}$	8900	2.00	2.90	1290	2.96	6.66	0.53	3990	12850
$\frac{5}{8}$	12390	2.59	4.86	2290	3.44	8.95	0.65	5350	22800
$\frac{3}{4}$	18580	2.96	6.22	3580	4.38	14.56	0.78	8420	27300

The elastic limit as found by these tests for bearing across the fibers of Douglas fir was from 410 to 680 lb. per sq. in. No information was given relative to the moisture content of the wood. Figure 3*a* shows a typical load compression curve. The elastic limit is reached at a compression of .04 to .05 in.

[1] Eng. News, vol. 71, page 666, March 26, 1914.

A series of 48 tests of ribbed cast-iron washers without rims was made in 1909 by H. M. Spandau in the civil engineering laboratory of Cornell University. All of them were designed for $\frac{3}{4}$-in. bolts. Their dimensions expresed in inches are given in the following table:

Dimensions of Special Cast-Iron Washers

Designation	Diameter	Height	Ribs		Base Plate Thickness
			Number	Thickness	
A	3	$\frac{5}{8}$	4	$\frac{3}{16}$	$\frac{1}{8}$
B	3	$\frac{5}{8}$	6	$\frac{1}{8}$	$\frac{1}{8}$
C	3	$\frac{5}{8}$	6	$\frac{1}{8}$	$\frac{1}{8}$
D	3	$\frac{5}{8}$	4	$\frac{1}{8}$	$\frac{1}{8}$
E	3	$\frac{11}{16}$	4	$\frac{1}{8}$	$\frac{3}{16}$
G	3	$\frac{9}{16}$	6	$\frac{1}{8}$	$\frac{3}{16}$
H	3.5	$\frac{5}{8}$	6	$\frac{1}{8}$	$\frac{1}{8}$
O	2.5	$\frac{9}{16}$	6	$\frac{1}{8}$	$\frac{3}{16}$
BB	4	1	6	$\frac{3}{16}$	$\frac{1}{4}$

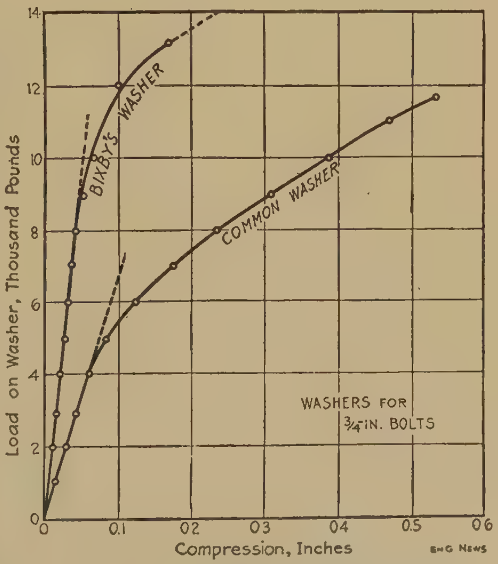


Fig. 3*a*.—Load-compression Curves for Cast-iron Washers.

In all cases the vertical cylinder or spools are $\frac{5}{16}$ in. thick. The net bearing area is 4.39 sq. in. for the $2\frac{1}{2}$-in. washers, 6.54 for the 3-in., 9.10 for the $3\frac{1}{2}$-in., and 12.05 for the 4-in. washers. The weights per hundred are as follows, in pounds: *A*, 53; *B*, 50; *C*, 49; *D*, 45; *E*, 55; *G*, 48; *H*, 56; *O*, 42; and *BB*, 135. Washers *C*, *E*, and *G* have the lower outer edges rounded to a radius of $\frac{1}{8}$ in. and washers

D have the upper surfaces of the ribs curved to a radius of $2\frac{7}{8}$ in. to improve their appearance. All the washers except *BB* are ordinary gray castings made in the Sibley College foundry as a part of the regular output, while washers *BB* are the standard of the Bridge and Building Department of the Atchison, Topeka, and Santa Fe System and similar to those of Table 2*c*.

Four kinds of timber were employed in the tests: Douglas fir, yellow poplar or whitewood, red oak, and white oak. The sticks were from $3\frac{1}{4}$ to $5\frac{1}{4}$ in. thick. The amount of compression was measured with precision for increments of 200 lb. in loading, except for washer *BB*, where the increments were 500 lb. Figure 3*b* gives a typical diagram indicating the relation between the load and the compression of the wooden stick under the washer. The tests were continued until the fibers of the wood broke or sheared from adjacent ones. The elastic limit was determined in every case.

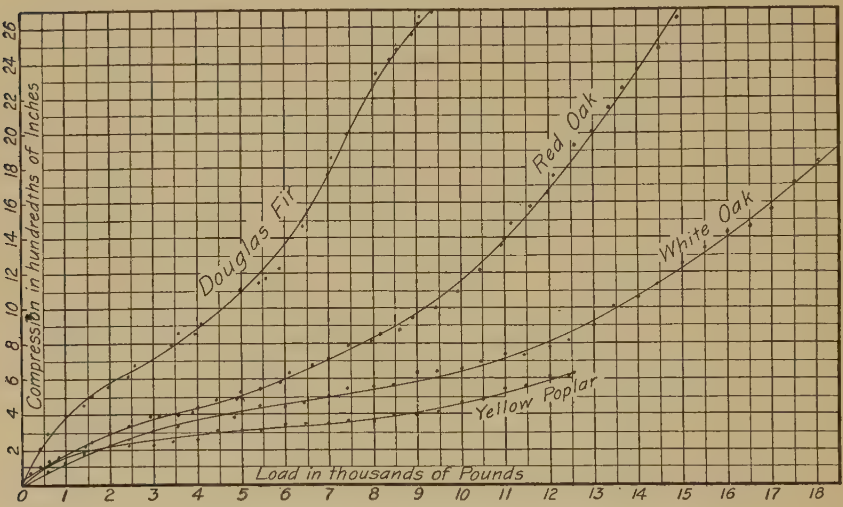


FIG. 3*b*.—Typical Curves for Compression under Washers.

Washers *A*, *D*, *H*, and *BB* were tested to destruction on red oak or white oak; also two specimens of *H* were broken on yellow poplar. In every case the failure occurred in the base plate between the ribs. The average ultimate resistances were 13,200, 12,250, 12,400, and 21,000 lb. respectively. Two specimens of

B failed, while one remained unbroken at 18,000 lb. when the wood gave way. The average for *B* is 14,250 lb. One specimen of *E* ruptured, while two remained intact when the wood yielded at 14,000 lb. The average for *E* is 13,450 lb. Washers *C*, *G*, and *O* failed to break in any test, although the loads averaged 15,500, 16,700 and 15,400 lb. respectively when the wood reached its ultimate strength.

Only two specimens with four ribs survived the tests, and these had base plates $\frac{3}{16}$ in. thick and rounded edges. Only one washer with a rounded edge failed. Washers *C* differ from *B* only in the rounded edge, but none of the former broke, while two of the latter broke under less load. Washers *G*, which differ from *C* by having a thicker base plate but a reduced height, are found to be stronger, thus indicating the effect of strengthening the base plate. These results show that washers *G* are the best proportioned for a diameter of 3 in., and all of them will support more than four times the allowable stress in the bolt. The three test loads for *G*, none of the washers breaking, are 15,000, 16,900, and 18,200 lb., the first one on red oak and the other two on white oak. The $3\frac{1}{2}$-in. washer *H* should have a $\frac{3}{16}$-in. base plate, its height remaining the same. Its edge should also be rounded, since this form permits a considerably higher load before crushing or cutting into the fibers. The tests also show that the ultimate resistance per square inch of the timber decreases as the area of the washer increases, except for yellow poplar, which is very tough wood. The same relation exists at the elastic limit with the exception of a few tests on yellow poplar. Three tests of washers *O* ranged from 15,300 to 15,600 lb., or an average of 3507 lb. per sq. in.

Four tests were also made of a malleable iron washer. Its diameter is 3 in., but the net bearing area is only 4.5 sq. in., on account of its peculiar form, which sacrifices an annular area near the spool in order to secure a wider top. The metal is closely $\frac{3}{32}$ in. in thickness. The washers did not fracture, but the plates bent up between the ribs, of which there were, however, only three instead of four as illustrated in Fig. 2*e*. The average resistance was 1300 lb.

Since the weight of an ogee washer for a $\frac{3}{4}$-in. bolt varies from 0.7 to 1.0 lb., these tests indicate that it is possible to design an equally strong washer with a saving of from 30 to 50 percent.

Tests were made on the types of cast-iron ribbed washers used at the Panama-Pacific Exposition, the dimensions being the same as given in Art. 2 except that for $\frac{3}{4}$-in. washers the value of h was $\frac{13}{16}$ in. instead of 1 in. and for both the $\frac{3}{4}$-in. and $\frac{7}{8}$-in. washers the value of t was $\frac{3}{16}$ in. instead of $\frac{1}{4}$ in. The washers were made at two foundries, "A" and "B." The results were as follows:[2]

Size, In.	From Foundry "A"		From Foundry "B"	
	Ultimate load, lb.	Type of Failure	Ultimate load, lb.	Type of Failure
$\frac{5}{8}$	....	....	18,000	
$\frac{5}{8}$	....	....	20,000	Through rib near rim
$\frac{3}{4}$	20,000	At edge of head.....	7,500	Through head
$\frac{3}{4}$	28,000	Through head.......	9,500	Through head
$\frac{3}{4}$	16,000	Through ribs, flaws..	9,500	At edge of head
$\frac{3}{4}$	17,000	Through ribs, flaws..		
$\frac{7}{8}$	33,000	Through head.......	16,000	At edge of head
$\frac{7}{8}$	33,500	At edge of head.....	14,000	At edge of head
1	23,000	Through head.......	25,000	Through head
1	....	....	17,000	
1	....	....	23,000	Small flaws
1	....	....	26,000	Small flaws
1	....	....	23,000	Small flaws

Tests made at Cornell University by L. R. Rodenhiser[3] on 6-rib washers indicate that washers with the following dimensions, in inches, will develop the strength of the bolt:

Size of Bolt	Diam. of Washer	Thickness of Plate	Thickness of Rib	Thickness of Spool
$\frac{1}{2}$	$2\frac{1}{4}$	$\frac{1}{8}$	$\frac{1}{8}$	$\frac{3}{16}$
$\frac{1}{2}$	$2\frac{3}{4}$	$\frac{1}{8}$	$\frac{1}{8}$	$\frac{3}{16}$
$\frac{3}{4}$	3	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{3}{16}$
$\frac{3}{4}$	4	$\frac{3}{16}$	$\frac{1}{8}$	$\frac{3}{16}$
1	$5\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{3}{8}$

[2] Eng. News, vol. 71, page 666, March 26, 1914.

[3] Cornell Civil Engineer, vol. 23, page 41, November, 1914.

The last washer has a rim $\frac{1}{4}$ in. thick. The others do not have rims.

PROBLEM 3. Determine approximately the stress per square inch at the elastic limit of the Douglas fir and red oak in the tests indicated graphically in Fig. 3*b*.

ART. 4. NAILS AND SPIKES

A nail consists of a slender body of metal, either tapering toward or pointed at one end, and with a head at the other end, used to drive through and into wood to fasten two pieces together, or to connect some other objects or materials in a similar manner.

Nails are classified both with respect to their method of manufacture and to their form or use. A cut nail is the common nail of rectangular cross section, which is cut from a metal strip by machinery, which also upsets and forms the head. It is called a cut nail to distinguish it from the wrought nail, which is forged either by hand or by machinery. The latter is also called a clinch nail, because its protruding point may be bent down readily without breaking, and is therefore secure against withdrawal due to shocks or vibrations. A wire nail is made directly from wire by machinery which forms both head and point.

According to their form or use, nails are known as common or ordinary, finishing, core, fence, casing, slate, light, brads, etc. Both cut and wire nails are now generally made of steel. Nails for special purposes in which corrosion, magnetic effect, and galvanic action must be avoided are made of copper, brass, and other alloys. The modern steel nails rust so much faster than the old wrought-iron nails that it is necessary to use copper nails to secure the same life as for the material connected when exposed to the weather or to other adverse conditions.

The length of nails ranges from 1 to 6 in., the successive sizes differing by $\frac{1}{4}$ in. up to $3\frac{1}{2}$ in. and by $\frac{1}{2}$ in. beyond that limit. A nail 3 in. long, is, however, not designated by its length, but as a tenpenny (written 10d) nail. This term is a corruption of "10 pounds of nail," meaning 10 pounds to 1000 nails, which indicates the method originally adopted for measuring them. This nomenclature has lost its original significance, for at present one manufacturer furnishes 72 common cut nails or 69 wire nails per pound, whereas another one furnishes 60 and 77 per pound respectively of the size known as tenpenny nails. The sizes run from twopenny to sixtypenny. The kinds and dimensions of nails are not only listed in the catalogs of manufacturers, but are given in various handbooks prepared for the use of architects, superintendents, inspectors, and builders.

The cut nail is driven so that its tapering sides bear against the bent ends of the fibers, thus acting like a wedge. The blunt point breaks and bends the fibers as it penetrates the wood. The wire nail is round, and has a pyramidal point, which separates the fibers when driven into the wood, and its resistance to withdrawal is therefore due to the bearing of a part of its cylinderical surface against the sides of the fibers. In some kinds of construction, as in freight-car building and repair work, it is found economical to use a pneumatic hammer to drive the nails after starting them by hand. In this way two men can do the work formerly requiring four men, when all of it was done by hand.

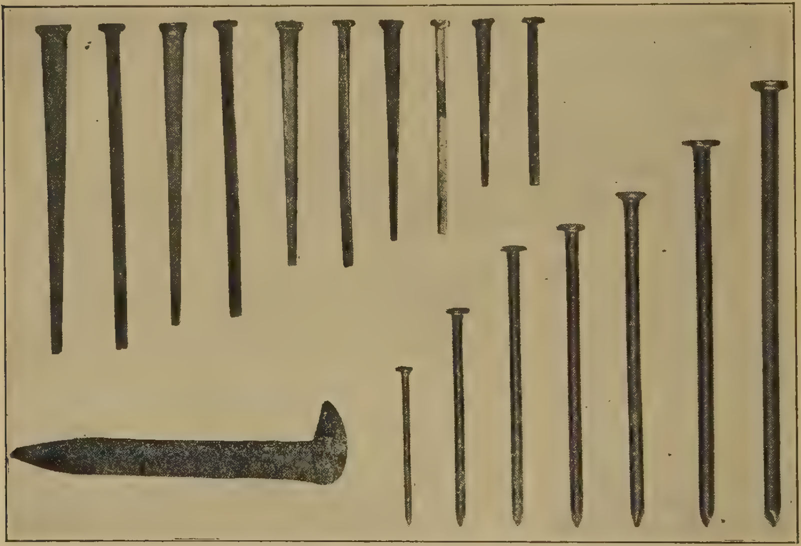
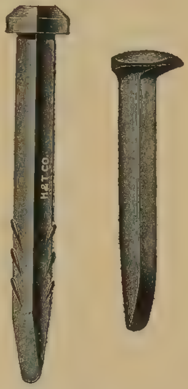


Fig. 4*a*.—Cut Nails, Wire Nails and Railroad Track Spike.

A patent cement-coated nail has been introduced in order to secure greater adhesion in the wood, and thus permit smaller sizes to be used than with the ordinary wire nail. They are made by coating wire nails with an asphaltum cement.

There are two principal classes of spikes used in timber construction; in one class the spikes have the same form as a nail, either cut or wire; those of the other class are called boat or ship spikes. The former are manufactured in the same manner as cut or wire nails, whereas the latter are forged from square bars of wrought iron or steel, and have a blunt chisel or wedge point. The length of the former ranges from $3\frac{1}{2}$ to 7 in. for the cut spike and from 3 to 12 in. for the steel wire spike, whereas that of the latter ranges from 3 to 16 in. The boat spikes have thicknesses of $\frac{1}{4}$, $\frac{5}{16}$, $\frac{3}{8}$, $\frac{7}{16}$, $\frac{1}{2}$, and $\frac{5}{8}$ in., whereas the length varies for each thickness, being from 3 to 7 in. for the smallest, and from 8 to 16 in. for the largest thickness. The point is wedge-shaped, the length of the taper being about twice the thickness of the spike (Fig. 4*b*). The edges should not be ragged, as indicated in the illustration, since it reduces the holding power (Arts. 5 and 7). The railroad track spike used to fasten the steel rail to a timber cross-tie resembles the boat spike in general form, except in its eccentric head, its largest size being $\frac{5}{8}$ in. thick and 6 in. long (Figs. 4*a* and 4*c*).



FIGS. 4*b* and *c*.—Boat Spike and Railroad Track Spike.

PROBLEM 4. Get a 20d and a 60d cut nail and measure the thickness and width at the point, at the middle, and directly under the head; also the diameter of wire nails of the same size.

ART. 5. HOLDING POWER OF NAILS

A carefully arranged set of experiments on the relative holding power of nails under different conditions was made by F. W. Clay in the laboratory of the College of Civil Engineering, Cornell University, the results of which were published in Engineering News, vol. 31, page 22, Jan. 11, 1894. The most important facts and principles obtained from these tests are given in the following paragraphs.

By driving a cut nail in a strip of wood between two holes it was found that the resistance to withdrawal is less than 8 percent smaller than when driven into the same stick in the usual manner.

As this difference is less than that frequently obtained for two nails driven under the ordinary conditions, it may be said that the holding power of a cut nail depends only on the friction on the wedge or taper sides. As the nail penetrates the wood the blunt point breaks and bends the fibers and then compresses them as it is driven deeper. On withdrawal, these fibers tend to straighten out and increase their pressure on the rough surface of the taper sides. The other sides of the nail are parallel and smooth and develop but little or no resistance.

Tests made on six different forms of point indicated their effect upon the holding power of the cut nail. The highest result was given by a short pyramidal point, the resistance being 32 percent greater than for the customary blunt end. This increase in holding power appears to be due both to a reduction in the number of fibers that are injured by breaking, and to separating the remaining fibers so as to develop pressure and friction on the parallel sides of the nail. If, however, this form of point is materially lengthened, it will merely separate and compress the fibers, thus eliminating the wedge action of the taper sides, and reduce the holding power below that for the blunt point used in practice.

If the ordinary cut nail is driven with its taper sides parallel to the fibers of the wood, the resistance to withdrawal is larger than when driven in the usual manner. This difference is much smaller in the hard woods than in the soft ones. The tendency to split the timber is materially increased at the same time.

The wire nail with its ordinary pyramidal point forces apart the fibers and develops pressure on the sides of the fibers, the surface of contact being less than the full cylindrical surface of the nail. If the wire nail had a blunt point, its holding power would be less than half as great. The blued surface which is slightly granular and rough increases the holding power and soon rusts fast to the wood, but the process employed injures the quality of the nail, so that many are spoiled in driving.

Comparative tests showed that the ratio of the initial holding power to the weight is larger in every case for the cut nail than for the wire nail. The effect of sharp barbs is to reduce the resistance of both cut and wire nails.

When the cut nail is withdrawn, its resistance decreases very rapidly. When withdrawn about 5 percent of its penetration, or fairly started, only about 60 percent of its holding power remains. Its holding power decreases more rapidly than that of the wire nail, and under vibration the cut nail jars loose sooner than the wire nail. The proper length for a nail to resist direct withdrawal is about 3 times the thickness of the thinner piece nailed.

The absolute value of the holding power of a nail depends not only upon the form, length, and kind of nail, but also upon the kind of wood, its state of seasoning, thickness of the annual rings, the proportion of hard summer wood in the rings, the relation of the nail to the rings and to the fibers, the mode of driving and of withdrawing the nail. On account of the large number of these elements and the considerable variation of some of them, any published tables of absolute holding power must be used with caution, especially when the conditions of the tests are not fully described.

As the result of a challenge by the wire-nail manufacturers to the cut-nail manufacturers, a series of experiments was made at the Watertown Arsenal in December, 1892, and January, 1893, including tests of 58 groups, each group including 10 cut nails and 10 wire nails of the same size. A summary of the results was published in the Railroad Gazette, vol. 25, page 356, May 12, 1893. At first forty groups were tested, including various sizes of common, light common, finishing, box, and floor nails, all driven in spruce wood, and it was found that the superiority of the cut nails over the wire nails in holding power for the classes mentioned was 47.5, 47.4, 72.2, 50.9, and 80.0 percent respectively, or an average of 60.5 percent for all of these tests. After that, 18 extra groups of box nails were tested in pine with different relations to the grain of the wood, and in this case the superiority ranged from 64.4 to 135.2 percent, or an average of 99.9 percent. The least value of 64.4 percent applies to nails driven parallel to the fibers.

Although these experiments apparently prove that in every case the cut nail was superior to the wire nail in holding power, it must be remembered that the holding power considered is the resistance at starting to pull the nail, and not that still remaining after the nail has been partially withdrawn, the basis of comparison being the area of the surface of the nail in contact with the wood. As subsequently shown in this article, the holding power decreases much more rapidly in the cut nail on account of its wedge form, and a comparison of that remaining indicates the superiority of the wire nail for most purposes. Experience has shown that boxes nailed with wire nails stand handling in transportation better than those made with cut nails, especially for the heavier commodities.

RESISTANCE AND WORK IN DRIVING AND PULLING NAILS IN YELLOW PINE

Size and Kind of Nail	Number of Nails Per Pound	Depth of Penetration in Inches	Maximum Load in Pounds for Driving	Maximum Load in Pounds at Start in Pulling	Work per Nail in Inch-Pounds		Ratio of Work in Pulling to Driving
					To Drive	To Pull	
20d cut	23	3.5	820	921	1523	478	0.32
20d wire	34	3.5	376	318	865	473	0.55
10d cut	70	3.0	342	357	585	201	0.36
10d wire	105	3.0	232	214	436	220	0.51
8d cut	88	2.25	312	328	419	140	0.33
8d wire	132	2.25	199	167	279	105	0.38
6d cut	168	1.75	221	156	274	65	0.24
6d wire	252	1.75	135	88	165	63	0.38

The work performed as well as the force required in both driving and pulling cut and wire nails were investigated by W. E. Barnes and R. O. Stillwell in the Sibley College laboratory of Cornell University, the detailed results of which may be found in a paper by R. C. Carpenter in Transactions American Society of Mechanical Engineers, 1895, vol. 16, page 1002.

Nails of different sizes were forced into a piece of southern pine, which was as nearly homogeneous as it was possible to obtain, by one of the heads of a testing machine, and the force required at the end of each one-quarter-inch of penetration was noted. In pulling them the force required was again noted at the beginning of each quarter-inch withdrawal. Ten nails of each size and kind were tested and the averages taken in plotting the diagrams.

A general summary of the results is given in the preceding table. From the table and diagrams here reproduced it will be noted that they confirm the previous statements based on other experiments with reference to the relative maximum holding power of cut and wire nails, and the rate of reduction in the resistance as the nails are withdrawn. They also show that the total work in inch-pounds per nail in driving cut nails is much more than for wire nails. The work required in pulling nails is, however, nearly equal for the two kinds of nails, being sometimes larger for one and then for the other. If the work per nail be multiplied by the number of nails per pound, it will be observed that the work required per pound of nails is less for wire than for cut nails in driving, but considerably greater in pulling. The ratio of the work of pulling to that of driving is also much higher for the wire than for the cut nail.

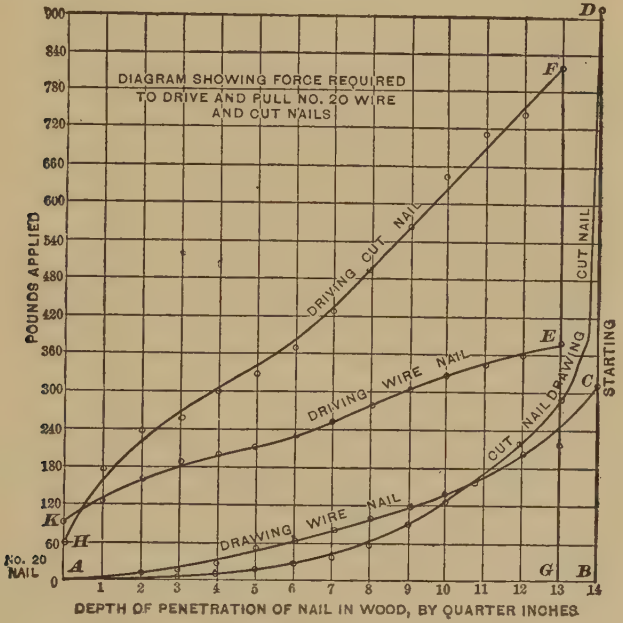


Fig. 5*a*.—Resistance to Penetration and Withdrawal of 20d Nails.

As previously stated, a nail in tension should have a length equal to about three times the thickness of the piece held in place by the nail. The holding power then depends upon two-thirds of the length of the nail from its point. The diagrams show that in this case the holding power is somewhat larger for the wire nail than for the cut nail, whereas the relative difference in the total work required to withdraw that part of the nail is still greater and also in favor of the wire nail. The student is referred

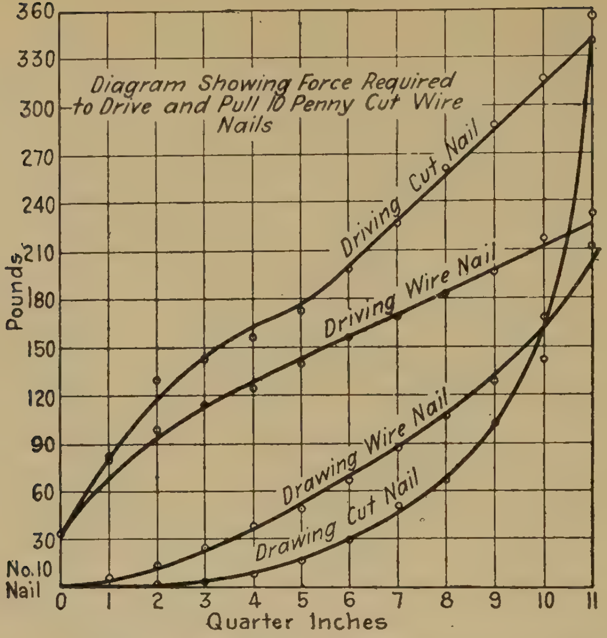


FIG. 5b.—Resistance to Penetration and Withdrawal of 10d Nails.

to the discussion of Carpenter's article, in which important facts and experience are given on the use of different kinds of nails for various purposes to which they are adapted.

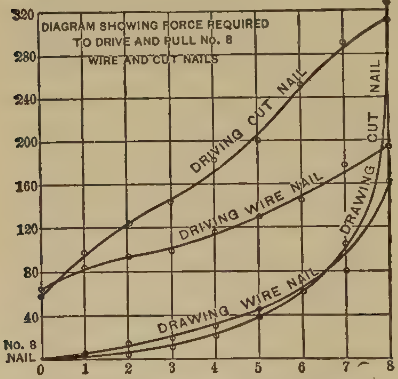


FIG. 5c.—Resistance of 8d Nails.

The results of a series of experiments on the holding power of nails in Douglas fir and redwood, made by Frank Soulé in 1896, were published in the Journal of the Association of Engineering Societies, vol. 20, page 115, February, 1898. A few of the more important conclusions are as follows: The cut nail is superior to the wire nail when driven into Douglas fir, but in redwood the wire nail is superior to the cut nail. The redwood is softer than the Douglas fir, but the former had 39 rings, whereas the latter had 14 rings per inch. This fact bears out the theory as to the manner in which the wire nail holds. The holding power of nails increases with time in redwood, but decreases in Douglas fir. In both kinds of wood the holding power is about three times as great when the nails are driven perpendicular to the fiber as when driven parallel to the fiber. In Douglas fir, cement-coated nails have about 30 percent greater holding power than plain wire nails, but in redwood the difference is small. The following average ultimate resistance with nails driven perpendicular to the fibers, expressed in pounds per square inch, were obtained: Cut nails in Douglas fir, 481; cut nails in redwood, 282; wire nails in Douglas fir, 296; wire nails in redwood (3 tests only), 367.

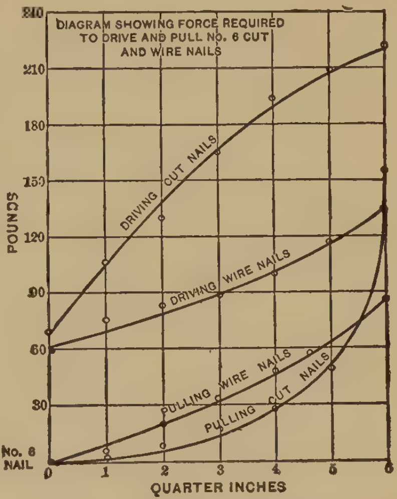


Fig. *5d.*—Resistance of 6d Nails.

A series of tests on the adhesive resistance of nails in five kinds of timber was made at the Watertown Arsenal. The average holding power for cut and wire nails when driven in the usual manner is given in the accompanying table:

Ultimate Holding Power of Cut and Wire Nails
(Expressed in pounds per square inch)

Kind of Wood	Direction of Nail to Grain	Cut Nails		Wire Nails 10d
		8, 9, 10, 20d	50, 60d	
White pine	Perpendicular	451	350	167
	Parallel	223	192	126
Southern pine	Perpendicular	685	636	318
	Parallel	578	483	252
White oak	Perpendicular	1277	1072	940
	Parallel	1134	977	802
Chestnut	Perpendicular	....	683	...

The holding power is expressed in pounds per square inch of the surface of contact between nail and wood. The depth of penetration used did not exceed two-thirds of the length of the nail, so as to conform to average conditions in practice. Four tests were made for each size of nail, except the 50d and 60d, for which the number was doubled.

It was found that when the cut nail is driven with the taper acting across the grain, thus tending to split the timber, the average resistance for all the sizes mentioned above is increased 33 percent in white pine, 21 percent in yellow pine, 14 percent in white oak and 5 percent in chestnut, or an average of 18 percent. The effect of barbing the cut nail is to diminish its average holding power in these four kinds of wood about 15 percent, the reduction being the least in the soft wood, like white pine.

When sticks of wood, in which 60d nails had been driven, were exposed to tide-water and air alternately for 13 days, until they were water-soaked as deep as the nails penetrated, the adhesive resistance of the nails was found to be reduced 43 percent in white pine and 23 percent in yellow pine and white oak. Nails of the same size were then driven into the wet sticks and stored in a cool, dry building for 6.8 years. Upon being tested, the resistance of the nails was still found to be 43, 53, and 9 percent below the original values for the three kinds of wood respectively.

The cement-coated nails described in the previous article have a much larger adhesive resistance than smooth wire nails. The following table gives the results of comparative tests made in 1902 at the Watertown Arsenal. The nails were driven across the grain into pine timber, the heads being allowed to project $\frac{1}{4}$ to $\frac{1}{2}$ in. Each value is the average of three tests.

COMPARATIVE HOLDING POWER OF SMOOTH AND COATED WIRE NAILS

Size	6d	8d	9d	10d
Depth driven	$1\frac{5}{8}$	2	$2\frac{1}{4}$	$2\frac{1}{2}$ in.
Cement-coated, holding power	226	316	327	418 lb
Common smooth, holding power	106	189	182	167 lb.
Ratio	2.13	1.67	1.80	2.50

The holding powers of wire nails in different kinds of wood may be assumed to be approximately proportional to their resistance to compression on the side of the fiber.

Dewell[4] gives the following values in pounds per square inch of metal surface in contact with the wood as the ultimate resistance to withdrawal for various types of fastenings. For designing purposes he recommends using values one-fourth of the ultimate.

ULTIMATE RESISTANCE TO WITHDRAWAL, POUNDS PER SQUARE INCH

	Southern Pine	Douglas Fir	White Pine	White Oak	Redwood
Cut nails, perpendicular to grain	500	500	300	1200	300
Cut nails, parallel to grain	300	300	275	1000	150
Wire nails, perpendicular to grain	300	300	170	900	300
Wire nails, parallel to grain	250	250	100	800	200
Boat spikes, edge of point parallel to grain	500	500	270	1000	
Boat spikes, edge of point across grain	370	370	200	750	

ART. 6. LATERAL RESISTANCE OF NAILS

The results of some experiments on the strength of wire nails to resist longitudinal shear in a lap joint (Art. 31) are shown in Table 6*a*. These tests include (1) those made in 1898–1899 by H. D. Darrow and D. W. Buchanan at Purdue University and described by W. K. Hatt in the Proceedings of the Indiana Engineering Society for 1900; (2) those made in 1907 by C. K. Morgan and F. Marish at Iowa State College and described by M. I. Evinger in Bulletin 2, Vol. 4, of the Experiment Station of that college; (3) those made at the Forest Products Laboratory, Madison, Wis., and described by T. R. C. Wilson in Engineering Record, Feb. 24, 1917, vol. 75, page 303; (4) those made by the Bureau of Buildings, Portland, Ore., and described in Engineering News-Record, Nov. 8, 1917, vol. 79, page 871; and (5) those made in 1897 by F. B. Walker and C. H. Cross at the University of Minnesota and described in the Journal of the Association of Engineering Societies, vol. 19, page 260, Dec. 1897.

[4] Handbook of Building Construction, vol. 1, page 245, Hool and Johnson.

TABLE 6*a*.—TESTS ON LATERAL RESISTANCE OF NAILS

Size	Diameter, Inches	Length, Inches	Southern Pine			Douglas Fir		Western hemlock	Southern Pine, Average	Douglas Fir, Average	Value of $P = 4000D^2$	Oak			Value of $P = 6000D^2$	Spruce	White Pine	White Pine to Norway Pine	Value of $P = 2500D^2$
			(1)	(2)	(3)	(2)	(4)	(4)				(1)	(2)	Average		(2)	(2)	(5)	
6d	.115	2	240	......	......	......	......	......	240	......	53	314	....	314	79	......	......	149	33
8d	.124	2½	361	294	......	478	......	......	327	478	61	454	656	555	92	137	230	189	38
10d	.148	3	724	324	......	475	525 407	519	524	469	88	762	608	685	131	237	235	246	55
12d	.148	3¼	......	625	......	360	......	......	625	360	88	....	665	665	131	332	318	......	55
16d	.165	3½	855	494	......	456	600 628	588	674	561	109	891	750	820	163	263	299	279	68
20d	.203	4	930	746	......	632	749 751	695	838	710	165	1350	992	1171	247	386	372	431	103
30d	.220	4½	......	1221	779 783	832	922 992	979	928	915	194	....	1392	1392	290	629	540	476	121
40d	.238	5	1450	1067	845 1125	837	1183 1023	....	1122	1014	226	1745	1580	1662	340	555	656	638	142
50d	.259	5½	......	1425	1261 1615	985	......	......	1434	985	268	....	1925	1925	403	563	712	800*	168
60d	.284	6	2000	1783	1144 1644	....	1800 873	....	1643	1336	323	1770	2125	1947	484	794	826	815*	202

(1) Darrow and Buchanan
(2) Morgan and Marish
(3) Wilson
(4) Bureau of Buildings, Portland
(5) Walker and Cross
* White pine to White pine

These tests include wire nails from 6d to 60d driven into southern pine, Douglas fir, western hemlock, oak, spruce, white pine, and Norway pine. In all cases the nails were driven perpendicular to the joint. The results are given in pounds per nail at the ultimate load.

In general, tests show that upon removing the load which causes a longitudinal extension of $\frac{1}{16}$ in. the joint recovers completely but if the extension exceeds that amount the recovery is only partial. This elongation occurs at about six-tenths of the ultimate load and represents the yield-point of the joint.

For the design of joints two types of formulas are used, one in which the safe load per nail is expressed in terms of the diameter squared of the nail and the other in terms of the penny weight. The first gives a much more nearly constant coefficient than the second.

For southern pine and Douglas fir the formula $P = 4000\ D^2$ has been proposed,[5] in which P is the safe design load in pounds for one nail and D is the diameter of the nail in inches. For southern pine this formula gives an average factor of safety for the accompanying tests of 5.4 and a minimum of 4.8. For Douglas fir these values are 4.9 and 3.7, and for western hemlock 5.1 and 4.2. For oak the coefficient in the formula may be taken at 6000 instead of 4000, in which case the average factor of safety is 4.9 and the minimum 4.0. If the coefficient for spruce, white pine and Norway pine is taken at 2500, the two factors of safety for spruce are 4.4 and 3.4, for white pine 4.6 and 3.6, and for white pine to Norway pine 4.4 and 3.9.

The tests of Walker and Cross showed that for ultimate loads the fibers on the adjacent surfaces of the timbers crush against the nail, which bends into a reverse curve. In no case did this crushing extend to the middle of the thinner timber. In oak the bends were very short and quite symmetrical. In the 20d nails they varied in length from $1\frac{3}{8}$ to $1\frac{1}{2}$ in. for the same extension or slip of $\frac{7}{8}$ in. in the joint. In such a joint, therefore, the nails are not subject to a true shear but rather to flexure. There seemed to be a fairly constant relation between the diameter of the nail and the length of bend.

In these tests if the nail penetrated the second timber less than half, but not less than one-third, of its length the ultimate strength

[5] Timber Framing, Dewell.

of the joint was reduced but the elastic limit remained the same. In experiments where the number of nails varied from 2 to 10, the strength of the joint was found to be proportional to the number of nails, thus indicating that they acted together in an effective manner.

In the tests on Douglas fir by the Bureau of Buildings, Portland, Ore., the first value (Table 6*a*) of each size of nail was obtained by nailing two side pieces, each with the grain of the wood vertical, to a center piece with the grain of the wood horizontal. For the second set of values, two side pieces, each with the grain of the wood vertical, were nailed to a center piece with the grain of the wood also vertical. The sizes of the side pieces varied from 1 by 6 in. to 3 by 8 in. and those of the center pieces from 4 by 6 in. to 6 by 10 in. All timbers were surfaced. The sizes of the pieces were such that the nails had a penetration in the center piece of 30 to 80 percent of the length of the nail, but the nails were used in single shear, in no case passing through the center piece. The number of nails used in each side piece varied from two to five. Each result is an average of from four to fourteen tests except that of the 60d nails where only one was made.

As a result of these tests the Bureau of Buildings established safe values—which seem high—for nails driven perpendicular to the grain in either Douglas fir or western hemlock, with the load perpendicular to the length of the nail, as follows:

Penny nail	10	12	16	20	30	40	50	60
Load value per nail, pound	120	120	160	200	270	320	400	480

These values fit closely the formula $P = 7d + 50$. For nails driven parallel with the grain of the wood it was recommended that the above figures should be reduced 25 percent. A reduction should also be made where the penetration in the holding piece is less than 50 percent of the length of the nail. The strength of the joint seemed to be fairly constant where the penetration was 40 percent or more, but with less penetration the loads were reduced. For a penetration of 30 percent the strength reduction amounted to 25 percent.

In the Wilson tests the piece through which the nails were driven, called the "cleat," was $2\frac{1}{2}$ in. thick and that receiving the nail, called the "block," was 5½ in. thick. The first figure in Table 6*a* for each size of nail is where the grain of both "cleat" and "block" are parallel to the direction of the load. For this arrangement, the elastic limit varied from 37 to 45 percent of the maximum load. The slip at elastic limit varied from .018 to .026 in., whereas the slip at maximum load varied from .54 to .70 in.

For the second set of results the grain of the "cleat" was at right angles to that of the "block" and the direction of the load. Here the elastic limit varied from 34 to 42 percent of the maximum load. The slip at the elastic limit varied from .028 to .039 in., whereas the slip at the maximum load varied from .88 to 1.32 in.

In the experiments of Darrow and Buchanan, to determine the effect of the length of nail, 14 southern-pine joints were tested, having 1⅛-in. cleats nailed to the blocks with 16d common wire nails cut to different lengths, varying by ¼ in. from 2 to 3½ in. At the yield point the resistance was 352, 392, 355, 410, 412, 415 and 367 lb., or an average of 386 lb. Each value is the mean of two tests. As the extreme range of these values is only one-half as large as the greatest difference between two tests for the same length, the strength at the yield point may be regarded as practically constant. The ultimate strength is nearly proportional to the length of the nail, decreasing a little in proportion for the longer nails, the average load per linear inch of nail being 238, 229, 213, 222, 217, 208 and 192 lb.

The effect of changing the diameter of nail but not the length was investigated with cleats ⅞ in. thick and wire nails 2 in. long. The results given below indicate that the larger sizes are relatively at a disadvantage.

Normal size of nail.............	6d	10d	16d	20d
Diameter, inches...............	0.112	0.15	0.16	0.187
At yield-point, pounds..........	224	352	417	402
Ultimate strength, pounds.......	327	462	507	660

The best angle at which to drive the nail is at right angles to the surface of contact between the timbers. On inclining the nails in either direction the resistance is reduced. The effect of barbing the nails and of soaking the joint is to decrease the strength of the joint.

In the tests above noted, in changing the direction of the grain so as to make it perpendicular to the direction of the applied load in either cleat or block, or in both of them, the resistance remained practically the same as when both cleat and block had the grain parallel to the load. When, however, the fibers of the block were so placed that the nails were driven parallel to the fibers, the resistance decreased at least one-third.

The strength of various sizes and kinds of steel nails in the Darrow and Buchanan tests is given in the accompanying table, some of which are also shown in the first table of this article. Two sticks of southern pine were used, having an ultimate compression strength parallel to the grain of 7000 and 10,200 lb. per sq. in., respectively, the former being used in testing all nails not larger than 8d, whereas the latter was used for the other sizes. The compressive strength of the oak was 5300 lb. per sq. in.

Lateral Resistance of Nails
(Expressed in pounds)

Kind of Nail	Size	Southern Pine		Oak		Kind of Nail	Size	Southern Pine		Oak	
		Wire	Cut	Wire	Cut			Wire	Cut	Wire	Cut
Common	2d	......	130	......	160	Common	60d	2000	1860	1770	.....
Common	3d	......	180	184	289	Finish	4d	106	163	186	262
Common	4d	198	213	211	344	Finish	6d	216	209	299	273
Common	6d	240	317	314	429	Finish	8d	264	282	359	405
Common	8d	361	427	454	573	Finish	10d	537	451	583	.....
Common	10d	724	932	762	822	Finish	12d	498	.....	637	.....
Common	16d	855	1079	891	1006	Fence	8d	690	686	709	780
Common	20d	930	1112	1350	1631	Fence	10d	855	912	1030	1092
Common	40d	1450	1360	1745	1874	Fine	3d	121	.....	164	.....

The strength was found to vary closely in proportion to the surface of the nail. At the yield-point the load per nail in pounds may be expressed as follows, A being the area of the surface in square inches: for cut nails in oak, $555A$; for cut nails in yellow pine, $430A$; for wire nails in oak, $315A$; and for wire nails in yellow pine, $255A$. The corresponding values of the ultimate strength are $785A$, $750A$, $540A$, and $420A$. These formulas fit the tests fairly well up to 40d nails, except for cut nails in pine, for which the limit is 20d.

In comparing the resistance per pound of nails it is found that the smaller nails hold more than the larger ones. The wire nails hold more than cut nails in 12 out of 14 cases, when driven into yellow pine, and in 8 out of 13 cases when driven into oak. The 10d common and finishing nails seem to be better proportioned than the 6d or the 8d.

In some experiments on nailed lap joints made by Clay (see reference in preceding article) it was found that the cut nail has a greater resistance than the wire nail in soft wood, while in hard wood the wire nail is superior, the difference being, however, comparatively small. He also found that the strength per unit of surface in the wood was approximately constant.

In Soulé's experiments (see preceding article) are included some tests on nailed joints, using both Douglas fir and redwood. It was found that the strength of the joint is practically the same for both kinds of wood, but that the cut nails develop 1.4 times the strength of the wire nails of the same nominal size.

It has been previously stated that the ultimate strength of the nail under a shearing load is proportional to the square of its diameter for a given ratio of its length to the thickness of the cleat; that it is also proportional to the area of contact with the wood; and that it increases in direct proportion to the length of the nail. These facts seem to indicate that the ultimate resistance is a combination of flexure and tension. Below the yield-point, however, it is probably pure flexure, but with an unequally distributed bearing on its surface, being the greatest next to the surface of contact between the timbers.

Some experiments on the dynamic strength of iron cut nails were made in the civil engineering laboratory of Lehigh University. With the nails to be tested a board was nailed to a heavy joist placed on a firm foundation so that the top of the board projected 3 or 4 in. above the joist. A wooden ram weighing 44 lb., fitting loosely in wooden guides, was used to find the dynamic energy required to break the nails in the joint.

The kind of wood into which the nails were driven seemed to make no difference in the dynamic strength developed. In order to compare the dynamic with the static strength the latter was observed for nails driven into both hemlock and oak. The thickness of the board was usually about one-third of the length of the nail. In the case of the smaller nails the heads were often pulled through the board under the static test. The results are briefly summarized in the accompanying table.

Ultimate Dynamic and Static Strength of Cut Nails in Shear

Size of nail	4d	6d	8d	10d	12d	20d
Number of tests	9	8	16	11	5	6
Dynamic strength, foot-pounds	13	21	35	38	67	90
Number of tests	10	6	6	13	7	2
Static strength in hemlock, pounds	185	276	302	416	366	679
Number of tests	7	3	7	9	3	2
Static strength in oak, pounds	287	419	390	439	524	685

The practical significance of these results may be illustrated by a single example. Neglecting losses due to impact, it takes about 35 ft.-lb. to break an 8d nail. If a man weighing 150 lb. dropped 3 ft. to the floor on a scaffold, he would break $\frac{450}{35}$ or nearly 13 nails, whereas the same number of nails could bear an ultimate static load equal to the weight of 26 men. The number of nails must be divided by a suitable factor in order to resist with safety the effect of the fall. This example may indicate the cause of accidents to workmen on scaffolds, to the construction of which too little thought is often given.

Problem 6*a*. Discuss the tests of nails in joints under a shearing load, and state the facts and arguments on which to base their comparative strength for different species of wood. On what physical property of the wood does the strength of nailed joints depend? Consider each property in turn, giving arguments for and against.

Problem 6*b*. Show the relations between the lengths of wire nails and their lateral resistance in oak wood with the aid of a diagram.

Art. 7. Holding Power of Spikes

A series of 27 tests was made on the holding power of common spikes at the Watertown Arsenal. All the spikes are $\frac{1}{2}$ in. in section and 8 in. long, including a wedge point 1 in. long, and were driven from 4 to 6 in. into the wood. The average resistance is given in pounds per square inch of the surface of contact between wood and spike exclusive of its wedge point.

ULTIMATE HOLDING POWER OF COMMON SPIKES

Size of Spike, Inches	Kind of Wood	Number of Tests	Resistance	
			Pounds per Square Inch	Pounds per Linear Inch
½	White pine	4	224	448
½	Yellow pine	8	530	1060
½	White oak	4	764	1528
½	Chestnut	3	720	1440

Two tests made by driving the spike parallel to the grain in white pine gave a resistance of only 70 lb. per sq. in. The remaining tests were made to determine the effect of water soaking the timber to the depth of penetration of the spikes. The resistance was reduced 40 percent in white oak, 31 percent in yellow pine, while in white pine no reduction was observed.

The accompanying table summarizes the results of some tests to determine the adhesive resistance of boat or ship spikes. The sizes tested are $\frac{3}{8}$, $\frac{7}{16}$, and ½ in., the spikes being driven 3, 4, 5, and 7 in. into white pine wood

ULTIMATE HOLDING POWER OF BOAT SPIKES PER LINEAR INCH

Size of Spike, Inches	Kind of Wood	Number of Tests	Adhesive Resistance, Pounds	Direction of Edge of Point Relative to Grain
$\frac{3}{8}$	White pine	18	370	Across
$\frac{3}{8}$	White pine	18	450	Parallel
$\frac{7}{16}$	White pine	10	344	Across
$\frac{7}{16}$	White pine	10	436	Parallel
½	White pine	2	429	Across
½	White pine	2	521	Parallel

The average resistance with the edge of the point parallel to the grain is about 23 percent greater than with the edge across the grain. When it is remembered that the allowable working stress on the side of the fibers is less than one-sixth of that on the end of the fibers, the relation given in the preceding sentence shows to what extent the fibers must be injured when the spikes are driven with the edge of the point across the fibers and no hole is previously bored.

To determine the resistance of a boat spike when pulled in the direction in which it is driven, 12 tests were made of $\frac{3}{8}$-in. spikes driven into a plank of white pine 3 in. thick until the head was flat with the surface. It required an average force of 1521 lb. to pull a spike with its head through the plank. The average resistance for 12 tests of $\frac{1}{2}$-in. spikes was similarly found to be 1708 lb.

Assuming that the resistance of the body of the spike in the direction of driving is 60 percent of the resistance to withdrawal, as determined for drift bolts (see Art. 10), the resistance due to the head of a $\frac{3}{8}$-in. spike is approximately $1521 - (3 \times 0.60 \times 370) = 855$ lb., whereas that for a $\frac{1}{2}$-in. spike is $1708 - (3 \times 0.6 \times 429) = 936$ lb.

The holding power of the common railroad spike in wooden cross-ties is given by W. K. Hatt in Circular 46 of the U. S. Forest Service. The size of the spikes used in the tests is $\frac{9}{16}$ in. square and $5\frac{1}{2}$ in. long under the head, being driven 5 in. into the wood in all cases. The average adhesive resistance was found to be as follows: for 5 tests in partially seasoned white oak, 6950 lb.; 5 tests in seasoned oak (probably red oak), 4342 lb.; 28 tests in seasoned loblolly pine, 3670 lb.; 12 tests in green hardy catalpa, 3224 lb.; 11 tests in green common catalpa, 2887 lb.; and 4 tests in seasoned chestnut, 2980 lb. A knotty cross-tie has about 25 percent less holding power for common spikes than one with clear wood. The circular also gives the results of tests with loblolly pine cross-ties which were steamed, or soaked, or steamed and treated with either creosote or zinc chlorid. The treating process reduced the holding power of the spikes.

Table 7*a* gives the results of a series of tests made by the American Railway Engineering Association to determine the effect of pre-boring holes for common railroad spikes with ordinary chisel points. Tests were made on $\frac{9}{16}$-in. holes in addition to those given in the table. The results shown for treated timber are for a 5-lb. creosote treatment. Tests also were made on a set of specimens having an 8-lb. treatment of a fifty-fifty mixture of creosote and crude oil. All spikes were driven until the heads were $\frac{7}{8}$ in. from the tie.

TABLE 7*a*.—HOLDING POWER OF RAILROAD SPIKES*

	$\frac{9}{16}$-in. by 6-in. Cut Spikes						$\frac{5}{8}$-in. by 6-in. Cut Spikes					
Diameter of hole, inches....	0	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{5}{8}$	0	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{5}{8}$
Untreated												
Red oak..................	7790	7920	7680	8380	7970	6360	8250	8410	8250	7750	7920	7530
Red gum..................	5380	5290	4760	5100	4580	3080	4950	5330	5090	4920	5700	4070
Black gum...............	6380	5280	5370	5750	6010	4350	5670	5930	5850	5290	6060	4890
Average.............	6520	6160	5940	6390	6190	4600	6290	6560	6400	5990	6560	5500
Treated												
Red oak..................	7460	7870	6890	7380	6310	5440	7340	7920	7160	6200	6380	5120
Red gum..................	5730	5140	4630	4390	4700	3160	4360	4270	4120	4000	3580	3230
Black gum...............	5090	5010	4730	4530	4480	2830	5490	5160	4940	4010	4960	3820
Average.............	6090	6010	5420	5430	5160	3810	5730	5780	5370	4740	4970	4060
Untreated												
Shortleaf pine.............	4310	4790	4440	4660	4550	3540	4680	4750	4430	4910	4670	4170
Loblolly pine.............	3080	2710	2820	3100	3130	2020	3040	3070	2890	2920	2830	2210
New Mexico pine..........	2490	2530	2740	2250	2030	840	3220	2910	3220	2690	2730	1640
Red spruce...............	4070	4170	4240	3870	4400	2140	5700	5470	5710	5880	5730	4110
Englemann spruce.........	1840	2430	2290	2080	2090	1010	2520	2210	2560	3280	2930	1940
Corbark fir...............	1690	1870	2050	2250	1850	960	2140	2520	2480	3130	2320	1710
Douglas fir...............	3600	3410	3510	3450	3560	2210	4440	4990	4840	4540	4060	4480
Average.............	3010	3130	3150	3090	3090	1810	3680	3700	3730	3910	3610	2890
Treated												
Shortleaf pine.............	3730	3870	3590	3310	3930	2680	3970	3990	4090	4120	4320	3420
Loblolly pine.............	3210	2990	2970	2940	3120	2310	3420	3270	3250	3670	3410	2450
New Mexico pine..........	3260	3740	3450	3110	2650	1160	4330	4140	4430	3780	3460	2430
Red spruce...............	4100	5140	4300	4040	3590	1620	4300	5550	5090	4430	4350	3100
Englemann spruce.........	1880	2520	2170	1930	1720	1060	2310	2150	2030	2150	2050	1540
Corbark fir...............	1940	2290	2200	1910	1920	1140	3130	2830	2650	2480	2440	1620
Douglas fir...............	3430	3000	2970	3050	3650	2050	4610	4290	4330	4230	3870	3280
Average.............	3080	3360	3090	2900	2940	1710	3720	3750	3690	3550	3410	2550

* Proceedings Am. Ry. Eng. Assoc., Bull. 301, page 209, Nov. 1927.

As a result of these tests the committee making the investigation recommended that holes for cut spikes in hardwood be $\frac{1}{16}$ in. smaller than the diameter of the spike and for soft woods $\frac{1}{8}$ in. smaller than the diameter of the spike. It was found that the use of $\frac{1}{4}$-in. and $\frac{3}{8}$-in. holes is impracticable due to the difficulty of of making the spike follow the hole. Tests also showed that even for the larger holes the chisel-pointed spike does not follow the hole as well as desirable and a point in which the chisel edge is slightly less than the diameter of the hole might be desirable.

Although these tests do not show any consistent increase in strength due to pre-boring holes, the efficiency and ultimate life of a spike in a tie depends upon the range of elastic behavior of the wood fibers and as pre-boring prevents much of the crushing and splitting of the wood fibers, a longer life probably results.

The lubricating effect of creosote reduces the holding power for all woods tested except New Mexico pine and loblolly pine. Here the creosote probably toughens the fibers.

Tests made by H. B. MacFarland[6] showed that a pyramidal point gives better results than the standard chisel point. The spikes tested were $\frac{9}{16}$ in. in section and approximately $5\frac{3}{4}$ in. long, the length of the point being 1.1 in. They were driven $4\frac{1}{8}$ in. into the ties. Where pre-boring was done the holes were $\frac{3}{8}$ by 4 in. The average holding power in pounds was as follows:

Average Holding Power, Pounds

	Hard Pine, Treated	Red Oak, Treated	White Oak, Untreated
Chisel point spikes, holes	3140	4810	5570
Pyramidal point spikes, holes	3770	5940	5995
Chisel point spikes, no hole	2215	4900	3600
Pyramidal point spikes, no hole	2975	5430	3230

Tests were also made on points composed of frustums of pyramids, the section at the tip being $\frac{1}{4}$ in. square and the length of the point varying from 0.5 to 1.7 in. In general the results were between those having pyramidal points and those having chisel points.

Tests made by the same investigator and recorded on page 790 of the above-mentioned Proceedings on $\frac{7}{8}$-in. screw spikes, screwed 5 in. into the ties, showed the following holding powers:

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[6] Proceedings Am. Ry. Eng. Assoc., vol. 15, page 766, 1914.

HOLDING POWER, POUNDS

Diam. of Pre-bored Hole Inches	Red Gum	Red Oak	Balsam	Long-leaf Pine	Short-leaf Pine	Douglas Fir	New Mexico Pine
$\frac{5}{8}$	7000	9055	7780	11970	7215	8550	6025
$\frac{11}{16}$	10310	11090	7295	10990	8355	8330	5193

These screw spikes had the rolled V-thread with $\frac{1}{2}$-in. pitch. The diameter at the bottom of the thread was $\frac{5}{8}$ in. and each spike weighed 19 oz. The average draw at which the maximum pull was reached was $\frac{5}{16}$ in. Most tests on common spikes have shown a maximum resistance at a small draw, usually less than $\frac{1}{16}$ in.

Tests made at the University of Illinois[7] on $\frac{7}{8}$-in screw spikes inserted 5 in. into $\frac{11}{16}$-in. pre-bored holes developed the following pulling resistances in pounds; treated blue ash, 9980; treated sweet gum, 8560; treated red oak, 13,600; treated elm, 12,310; treated poplar, 9960; untreated white oak, 12,520; and untreated chestnut, 7720. These screw spikes were $5\frac{1}{2}$ in. long, the diameter of the core being $\frac{11}{16}$ in. and the pitch $\frac{1}{2}$ in. The tests indicated that the holding power varies directly with the depth of penetration. Tests made under comparative conditions showed that the holding power of screw spikes was from 32 to 147 percent larger than that of common track spikes, varying as follows: poplar, 32; sweet gum, 56; ash, 65; chestnut, 67; red oak, 76; water oak, 79; elm, 83; white oak, 88; black oak, 103; beech, 138; and loblolly pine, 147.

A discussion of the use and advantages of screw spikes with numerous illustrations is given in Bulletin 50 of the Bureau of Forestry, issued in 1904.

ART. 8. LATERAL RESISTANCE OF SPIKES

Only a few tests results are available on the lateral resistance of wire spikes. Tests made by Walker and Cross as a part of the series abstracted in Art. 6 show the lateral strength of 80d—diameter .303 in. and length 7 in.—to be 895 lb. when fastening white pine to white pine and 1030 lb. when fastening Norway pine to white pine. For 100d nails—diameter .366 in. and length 8 in.—the respective values were 1291 and 1315 lb. These values indicate that the formulas recommended in Art. 6 for nails are also applicable to wire spikes.

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[7] Bulletin 6, University of Illinois Engineering Experiment Station.

The same investigators found that $\frac{3}{8}$-in. boat spikes driven through $2\frac{3}{4}$-in. white pine into white pine developed a maximum resistance of 1845 lb. for 7-in. lengths and 1955 lb. for 8-in. lengths.

On railroad curves the track spikes which hold the outer rail in position are subjected to lateral pressure due to the centrifugal force of the moving train, and to other forces caused by the tendency of locomotive or car trucks to continue in a straight line. In case tie plates with shoulders are used the lateral pressure is resisted by the spikes on both sides of the outer rail, but when no tie plates are employed it is resisted only by the spikes on the outside of the rail. In either case the friction between base of rail and cross-tie helps to resist the lateral pressure.

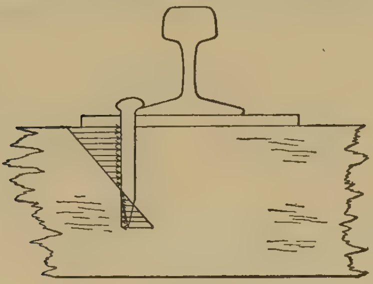


Fig. 8*a*.

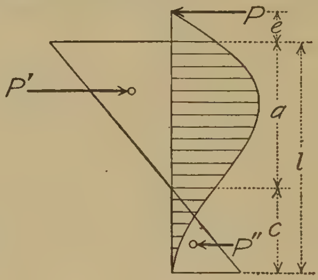


Fig. 8*b*.

The horizontal force applied at the edge of the rail flange causes a compression of the fibers of the wood on the opposite side of the spike above and on the same side below as indicated in Fig. 8*a*. The maximum compression occurs at the surface of the cross-tie, and since the deflection of the spike is comparatively slight for safe working conditions, the pressure on the wood may be regarded to vary the same as the ordinates to a straight line.

To deduce a formula expressing the relation between the rail pressure P and the maximum unit compression s on the fibers of the cross-tie, let e be the distance from the bearing of the rail flange in the spike to the surface of the cross-tie, l the penetration of the spike in the cross-tie, and let a and c be the depths of the wood surface in compression on opposite sides of the spike (Fig. 8*b*).

The resultants P' and P'' are proportional to the two unshaded triangular areas, respectively, or $P' : P'' = a^2 : c^2$, whence $P' = P''a^2/c^2$. As the sum of the horizontal forces must equal zero, there follows the relation $P' = P + P''$. The diagram also shows that $l = a + c$. Taking moments about some point on the line of action of P,

$$P''(l + e - \tfrac{1}{3}c) - P'(\tfrac{1}{3}a + e) = 0$$

Substituting the value of P' given above, replacing a by $l - c$. and reducing, there follows

$$c = \frac{l(l + 3e)}{3(l + 2e)} \text{ and } a = l - c = \frac{l(2l + 3e)}{3(l + 2e)} \tag{1}$$

Designating the width of the spike by b, the resultant P' equals $\frac{1}{2}\, abs$, and remembering that $P = P' - P''$, there is obtained

$$P = sbl^2/(4l + 6e), \text{ or } s = P(4l + 6e)/bl^2 \tag{2}$$

For example, let the spike be $\frac{5}{8}$ in. square in section and 6 in. long over all, or $5\frac{1}{2}$ in. below the head. Let the tie plate be $\frac{1}{2}$ in. thick and the edge of the rail flange $\frac{1}{4}$ in. thick; then $b = \frac{5}{8}$ in., $l = 4\frac{3}{4}$ in., and $e = \frac{5}{8}$ in. Substituting these values in the preceding formulas, the following values are obtained; $a = 3.002$ in., $c = 1.748$ in., $P = 0.620\,s$, and $s = 1.613\,P$. For a working unit stress on the ends of the fibers of 1200 lb. per sq. in., the lateral resistance of one spike is $P = 0.620 \times 1200 = 744$ lb. The ordinates to the curved line in Fig. 8*b* represent the bending moments in the spike. The section where the shear is zero is 1.25 in. below the surface of the cross-tie, and hence the maximum bending moment is $M = 744 \times 1.875 - 744 \times 0.680 = 889$ in.-lb. The lever arm 0.680 in. in this equation is the distance to the section from the center of gravity of the trapezoidal load area above the section as indicated in Fig. 8*b*. Equating this value of the bending moment to the resisting moment of the spike, the unit stress in the outer fiber is found to be 21,800 lb. per sq. in.

For a practical application of these formulas see an article on Stresses in Track Fastenings, by W. C. Cushing, including one by E. E. Stetson on A Study of Rail Pressures and Stresses in Track Produced by Different Types of Steam Locomotives when rounding Various Degree Curves at Different Speeds, published in Proceedings of the American Railway Engineering and Maintenance of Way Association, vol. 10, page 1431.

Tests made on the lateral deformation of track spikes as a part of the series abstracted in Table 7*a*, using the same timbers as these noted, showed no great variance between load and deflection with no hole, and load and deflection when the spike is driven into a $\frac{5}{8}$-in. hole. For $\frac{9}{16}$-in. by 6-in. spikes driven into untreated hardwood ties the average lateral resistance at the yield point varied from 2800 to 3050 lb. for the various sizes of pre-bored holes, being a maximum for the $\frac{7}{16}$-in. hole. For creosoted material the values ranged from 2350 to 2600 lb. For soft-wood untreated ties the range was from 2250 to 2400 lb. and for treated material, 2000 to 2400 lb. For $\frac{5}{8}$-in. by 6-in. spikes the values were somewhat higher. In most cases the lateral deflection at yield point was from .10 to .20 in.

Numerous impact tests to determine the relative efficiency of ordinary railroad and screw spikes to resist lateral displacement were made at the University of Illinois Engineering Fxperiment Station, a record of which was published in the second part of Bulletin No. 6, by R. I. Webber. The results show that the screw spike has a higher efficiency than the $\frac{9}{16}$-in. ordinary spike in all but two of the eleven kinds of wood used, and higher than the $\frac{5}{8}$-in. spike in all but three kinds of wood. The screw spikes had a core diameter of either $\frac{11}{16}$ or $\frac{21}{32}$ in.

PROBLEM 8. Find the lateral resistance of $\frac{9}{16}$-in. spike, 5 in. long under the head and without a tie plate under the rail, the allowable compression in the cross-tie being 1000 lb. per sq. in. Compute also the unit-stress in the outer fiber of the spike.

ART. 9. DRIFT BOLTS

A drift bolt is a piece of round or square iron or steel, of specified length, with or without head or point, driven as a spike. Different forms of head and point are shown in Fig. 9*a*. Before driving the bolt a hole must be bored somewhat smaller than the drift bolt. The best relation between diameters of bolt and hole is discussed in the next article.

A common form of drift bolt differs but little from a boat spike, except in length. A drift bolt acts like a nail, not only preventing lateral movement of the parts connected, but also their separation in a direction parallel to its axis. A dowel only prevents lateral displacement and is usually thicker and shorter than a drift bolt. Drift bolts should be long enough to give a good hold in the last timber which they are to penetrate.

Figure 41*g* in Art. 41 illustrates the use of drift bolts in fastening the cap to the piles of a trestle bent, and in Fig. 77*b* in Art. 77 they are used to fasten the caps to the posts, as well as the sill to the piles. They are designated spikes on the latter drawing. In one-half of the bent which has brick footings, drift bolts are also employed to fasten the sill to the posts, whereas in the other half dowels are used for this purpose.

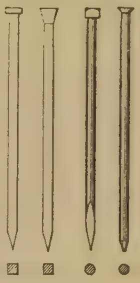
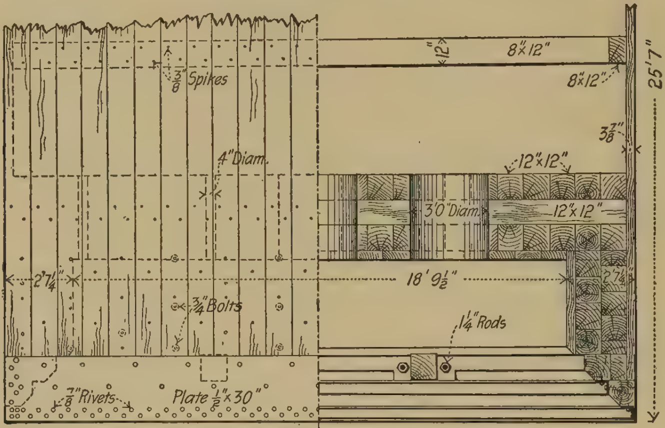


FIG. 9*a*.
Drift Bolts.

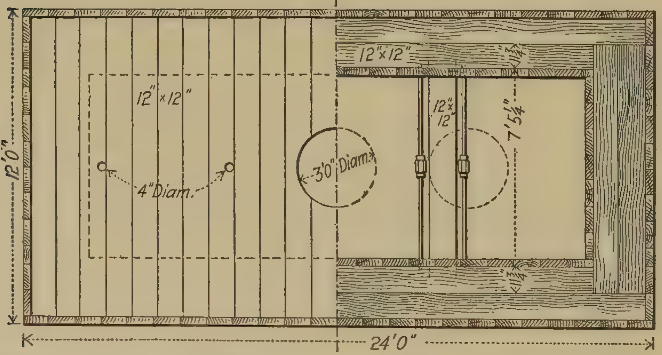
In a report on various methods of framing trestle bents a Committee of the American Association of Railway Superintendents of Bridges and Buildings made the following reference to drift bolts: "The third method used is by means of drift bolts, which act exactly as a nail or spike. They not only prevent any lateral movements in the timbers, but prevent the contiguous faces from drawing apart, by the friction of the wood on their sides. Drift bolts are very much the simplest and least expensive of the various types of fastenings used, and were it not for the same disadvantages (see Art. 15) as found in the use of dowels, they would be more generally used. However, this method is very common, and for all temporary work it is the best and most rigid manner of securing a strong joint at so slight a cost."

In the discussion of that report attention was called to the fact that large heads on drift bolts are objectionable since they crush the fibers on being driven into the wood, and in the cavities so formed the rainwater accumulates and causes decay; and that the difficulty of relining a stringer which is so fastened to a cap is materially increased. It was stated that by using drift bolts with chisel points and no heads, and using augers of smaller diameter than the bolt, sufficient head is formed in driving to answer all purposes, while facilitating the economic relining or renewal of any part. In some standard designs the head of the drift bolt is driven $\frac{3}{4}$ in. below the surface and the cavity filled with pitch to prevent the entrance of water. When drift bolts are used in trestle construction, their diameter is usually $\frac{3}{4}$ in. and sometimes $\frac{7}{8}$ in.

Figures 9*b*, *c*, and *d* show the use of drift bolts in fastening the timbers of the sides and roof of a pneumatic caisson for a building foundation. They are ¾ in. in diameter and 30 in. long. Fig. 9*e* indicates how the various courses of timber in a dam are connected by drift bolts. In the solid walls of 12 by 12-in. timbers of the cribs above the caissons of the Ohio River bridge at Cairo, Ill., the drift bolts were ⅞ in. square and 30 in. long, those driven into each course of timber being spaced 3 ft. apart. The largest crib was 30 ft. wide and 70 ft. long. In the large open cribs of the


⅜" Spikes
4" Diam.
2'7¼"
¾" Bolts
⅞" Rivets
Plate ½"×30"
12"
8"×12"
8"×12"
3⅞"
12"×12"
3'0" Diam.
12"×12"
18'9½"
1¼" Rods
2'7¼"
25'7"

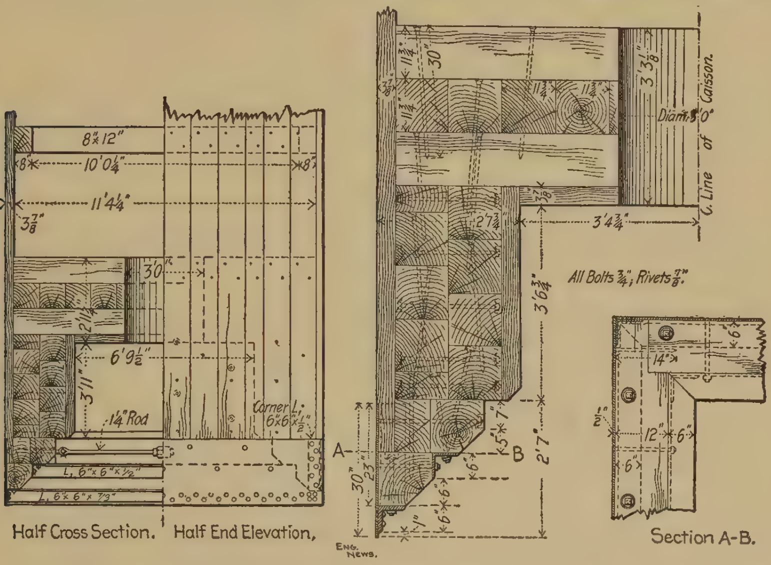
Half Side Elevation. Half Longitudinal Section.


12'0"
12"×12"
4" Diam.
3'0" Diam.
12"×12"
12"×12"
7'5¼"
3¼"
24'0"

Eng. News. Half Deck Plan. Half Plan of Working Chamber.

Fig. 9*b*.—Pneumatic Caisson for Foundations of Gillender Building, New York City. See *Eng. News*, vol. 37, page 13, Jan. 7, 1897.

Poughkeepsie bridge foundations, 1-inch drift bolts 30 in. long were used, and spaced about 1½ ft. apart. The largest crib was 60 by 100 ft., with six transverse and one longitudinal dividing walls between the compartments. In the construction of cribs it is in most cases the best plan simply to lap the timbers at the corners as well as elsewhere, and depend upon the drift bolts to hold them in place.


8"x12"
10'0¼"
11'4¼"
3⅞"
30"
6'9½"
3'11"
1¼" Rod
Corner L, 6x6x½"
L, 6"x6"x½"
L, 6"x6"x7/8"
Half Cross Section.
Half End Elevation,
A-
B
30"
23"
2'7"
3'6¾"
3'4¾"
2'7¾"
3'3⅛"
Diam. 5'0"
Line of Caisson.
All Bolts ¾", Rivets ⅞"
14"
12"
6"
Section A-B.
Eng. News.

FIGS. 9*c* and *d*.—Framing and Enlarged Details of Pneumatic Caisson.

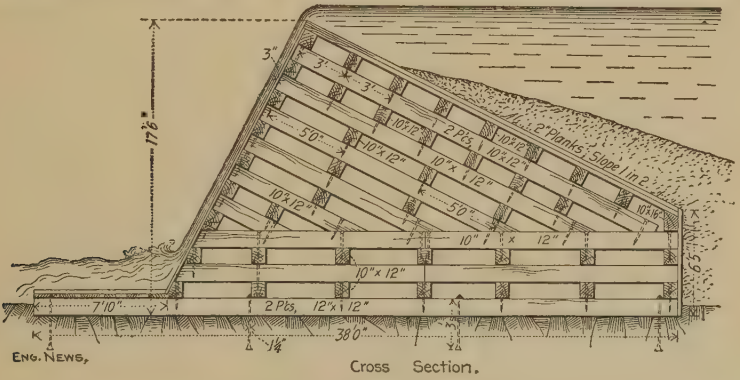

3"
3'
17'6"
5'0"
10"x12"
2 Pcs, 10"x12"
2" Planks; Slope 1 in 2.
10"x16"
10" x 12"
7'10"
2 Pcs, 12"x12"
38'0"
1¼"
6'5"
Eng. News,
Cross Section.

FIG. 9*e*.—Cross-section of Bear River Dam.

The plan of simply lapping the large timbers of cribs and caissons and depending upon the drift bolts to hold them in place has been strongly commended by experienced engineers.

Problem 9. Refer to Proceedings American Railway Engineering and Maintenance of Way Association, 1905, vol, 6, page 43, and one of the folding plates accompanying the article by A. F. Robinson. Make a drawing of the two drift bolts shown to one-half size and add the note of instruction.

Art. 10. Holding Power of Drift Bolts

The most systematic investigation of the adhesive power of round drift bolts was made by J. B. Tscharner in the engineering laboratory of the University of Illinois. The results were originally published in the Technograph, a local technical magazine, 1889–1890, No. 4, page 53, and a brief summary was given in Engineering News, vol. 25, page 199, Feb. 28, 1891. Rods or drift bolts of steel 1 in. in diameter were used, the diameters of the holes bored were $\frac{15}{16}$, $\frac{14}{16}$, $\frac{13}{16}$, and $\frac{12}{16}$ in. respectively in both yellow pine and white oak timber, and the total number of tests was 126. The ends were not pointed, but the sharp edges were hammered down so as not to cut the timber.

The results of 45 tests, in which the rods were driven from 3 to 12 in. into yellow pine, with holes $\frac{15}{16}$ in. in diameter, prove that the adhesion is directly proportional to the depth to which the bolt is driven.

Ultimate Holding Power per Linear Inch of 1-Inch Round Drift Bolts

Diameter of hole, inch	$\frac{15}{16}$	$\frac{14}{16}$	$\frac{13}{16}$	$\frac{12}{16}$
Holding power, pounds				
White oak	1300	1778	2500	1133
Yellow pine	375	633	788	400
Ratio	3.5	2.8	3.2	2.8
Ratio as percent:				
White oak	52	71	100	45
Yellow pine	48	80	100	51

The preceding table indicates that the holding power in white oak is approximately three times as great as in yellow pine. It also shows that the holding power for the $\frac{13}{16}$-in. holes is greater than for any of the others in both kinds of wood. As the holding power diminishes very rapidly for the next smaller size of the hole, it is apparent that the elastic limit of the wooden fibers is passed. Therefore to develop the maximum holding power of the bolt the diameter of the hole should be about 80 percent of that of the bolt, or the section area of the hole should be about 66 percent of that of the bolt.

The results given in the following table are the average values for 6 tests in yellow pine and 3 tests in white oak for every size of hole. The drift bolts were driven to a depth of 6 in. in every case.

RELATIVE HOLDING POWER OF DRIFT BOLTS WHEN PERPENDICULAR AND PARALLEL TO THE FIBERS

Diameter of hole, inch	$\frac{15}{16}$	$\frac{14}{16}$	$\frac{13}{16}$	$\frac{12}{16}$
Holding power, pounds:				
Yellow pine:				
Perpendicular	375	633	788	400
Parallel	200	280	344	222
Ratio	1.9	2.3	2.3	1.8
White oak:				
Perpendicular	1300	1778	2500	1133
Parallel	617	817	1033	867
Ratio	2.1	2.2	2.4	1.3

The data show that the holding power for drift bolts driven perpendicular to the fibers is approximately double that when driven parallel to the fibers. The average ratio is practically the same for both kinds of wood.

It was also found that in pulling a drift bolt the holding power decreases very rapidly as soon as motion begins, and that if originally driven 6 in. into the timber, only about 20 percent of the initial resistance remains when it is withdrawn 1 in., after which it decreases directly with the distance drawn out, during the following year.

The tests described above were continued by J. H. Powell and A. E. Harvey, who used 1-in. square drift bolts and yellow pine timber. Five tests were made for each size of hole, and the bolts were also driven to a depth of 6 in. The average holding power per linear inch was found to be as follows: for a 1-in. hole, 662 lb.; for a $\frac{15}{16}$-in. hole, 710 lb.; for a $\frac{14}{16}$-in. hole, 777 lb.; and for a $\frac{13}{16}$-in. hole, 675 lb. The largest value is given by the $\frac{14}{16}$-in. hole, which has a section area equal to 60 percent of that of the bolt, or nearly the same ratio as for round bolts.

After the bolts were withdrawn and the timbers were split open to examine the condition of the wood around the holes, it was found that in those larger than $\frac{14}{16}$ in. only the corners of the bolts had held effectively, whereas in the smaller holes the wood fibers were crushed and torn.

Although the section area of the 1-in. square drift bolt is 33 percent larger than that of the 1-in. round bolt, its maximum holding power is practically about the same. It must therefore be concluded that round drift bolts are more economical than square ones.

A series of 43 tests was made at the Watertown Arsenal, the results of which were recorded in Tests of Metals, 1902, page 577. The diameters of the bolts are 1, $\frac{7}{8}$, $\frac{3}{4}$, $\frac{5}{8}$, and $\frac{1}{2}$ in., respectively. In the Douglas fir wood five sizes of holes were bored for each size of bolt, the largest diameter in each case being $\frac{1}{16}$ in. less than that of the bolt, the other holes decreasing successively by $\frac{1}{16}$ in. In the white oak only 3 or 4 sizes of holes were bored for each size of bolt. The largest adhesive resistance for each size of bolt is given in the following table. The resistance per square inch relates to the surface of adhesion between the bolt and the timber.

Ultimate Holding Power of Round Drift Bolts

Diameter of bolt, inch	1	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{5}{8}$	$\frac{1}{2}$
In Douglas fir:					
Diameter of hole, in.	$\frac{13}{16}$	$\frac{5}{8}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{3}{16}$
Resistance per linear inch, pounds	2250	1901	1566	1555	1185
Resistance per square inch, pounds	724	696	656	762	741
In white oak:					
Diameter of hole, in.	$\frac{13}{16}$	$\frac{11}{16}$	$\frac{5}{8}$	$\frac{7}{16}$	$\frac{3}{16}$
Resistance per linear inch, pounds	3250	2588	1950	2462	2114
Resistance per square inch, pounds	1050	947	816	1210	1320

The averages of the resistance per square inch given above are 716 lb. for Douglas fir and 1068 lb. for white oak, the ratio being 1.49. The corresponding averages, when all sizes of holes are considered, are 539 lb. for Douglas fir and 957 lb. for white oak, the ratio being 1.78. These values are respectively 73.0 and 74.3 percent of the general averages for the lag screws tested at Watertown under the same conditions (see Art. 13). The resistance per square inch in the white oak is 1030 lb. for the 1-in. bolt in a $\frac{7}{8}$-in. hole, 947 lb. for the $\frac{7}{8}$-in. bolt in a $\frac{3}{4}$-in. hole, 789 lb. for the $\frac{3}{4}$-in. bolt in a $\frac{9}{16}$-in. hole, 1200 lb. for the $\frac{5}{8}$-in. bolt in a $\frac{1}{2}$-in. hole, and 1280 lb. for the $\frac{1}{2}$-in. bolt in a $\frac{1}{4}$-in. hole; thus showing that a variation of $\frac{1}{16}$ in. in size of the hole reduces the adhesion slightly from the maximum value for each bolt.

By comparing the resistance per linear inch for the 1-in. bolt in Douglas fir with the corresponding resistance given in the first table in this article, it is evident that the yellow pine used was not the southern longleaf species.

The following average results are computed from the record of earlier experiments made on drift bolts at the Watertown Arsenal. The sizes of bolts tested are $\frac{1}{2}$, $\frac{5}{8}$, and $\frac{3}{4}$ in. in diameter, and the depth to which they were driven varies from 4 to $5\frac{1}{4}$ in. The maximum holding power for each kind of timber is given in pounds per square inch of surface of contact between bolt and wood. The average holding power for 11 tests in cedar is 143; for 8 tests in chestnut is 365; for 2 tests in yellow pine is 397; and for 5 tests in white oak is 795 lb. per sq. in. A few tests were also made which indicate that the resistance is diminished about two-thirds by lubricating the wood with block oil or coating the bolt with asphalt varnish or with mastic and white zinc.

An extensive series of tests was made by A. Noble and C. P. Gilbert, U. S. Assistant Engineers. The total number of tests made was about 500, but the results indicate a considerable range both for the same stick and in different sticks of white pine used at both locations where the tests were made. For example, the holding power for a 1-in. round drift bolt in a $\frac{3}{4}$-in. hole in 8 different sticks varied from 731 to 1108 lb. per linear inch, each value being the mean of from 4 to 6 tests, while the range for single tests was still greater. In a few cases the highest value for one stick was more than double the lowest one for the same stick.

In general, the same relations between sizes of holes and bolts and between round and square drift bolts were obtained as those previously described in this article. The following table gives the average ultimate holding power per linear inch for 1-in. round drift bolts in white pine when tested as soon as driven. The respective means of over 150 tests of different conditions

HOLDING POWER PER LINEAR INCH OF 1-INCH ROUND DRIFT BOLTS IN WHITE PINE

Diameter of hole, inch......	$\frac{11}{16}$	$\frac{12}{16}$	$\frac{13}{16}$	$\frac{14}{16}$
Holding power, pounds......	837	856	899	792
Number of tests............	18	42	25	5

indicate that the resistance is about 10 percent greater when tested 7 months after driving. It was also found that a hole $\frac{14}{16}$ in. in diameter developed the greatest holding power for a 1-in. square drift bolt, the mean value for 48 tests being 901 lb. per lin. in. Some experiments were made on $1\frac{1}{8}$- and $\frac{3}{4}$-in. round and $\frac{5}{8}$-in. square drift bolts which show that practically the same relations hold between the section areas of the hole and bolt as those previously given.

The holding power for different sizes of drift bolts in timber of the same quality is closely proportional to the surface of contact between the bolt and the timber. Since the principal part of the resistance is due to the bearing of the side of the bolt on the ends of the wooden fibers, the holding power in different kinds of timber is approximately proportional to the compressive strength of short blocks for straight-grained clear wood.

The mean of 150 tests under various conditions shows that it requires only about 60 percent of the power to pull the drift bolt through the timber in the direction in which it was driven as to withdraw it in the opposite direction. It is also found that smooth rods have a greater holding power than ragged ones when pulled in either direction. The resistance is reduced a little more than 25 percent by moderate ragging, and over 50 percent by excessive ragging, these values being the average of 50 tests. When the rods were threaded, however, as in lag screws, the resistance to withdrawal is increased over 50 percent.

In some experiments made during the construction of the foundations of the Brooklyn bridge, it was found that a 1-in. round rod driven into a $\frac{7}{8}$-in. hole in longleaf yellow pine developed a holding power of 1250 lb. per lin. in. When the wood was very heavy and contained more pitch, the resistance was about 10 percent greater, but in lighter wood which contained less pitch it was about 20 percent less. In similar experiments at the Poughkeepsie bridge, the holding power of 1-in. round drift bolts driven into a $\frac{3}{4}$-in. hole in white pine and in Norway pine was 830 lb. per lin. in.

PROBLEM 10. With the aid of a diagram showing sections of drift bolts, and of the holes bored in the timber, as well as the direction of the fibers, it is required to give a rational explanation why the holding power of a square drift bolt is less than that of a round one of the same section area.

ART. 11. WOOD SCREWS

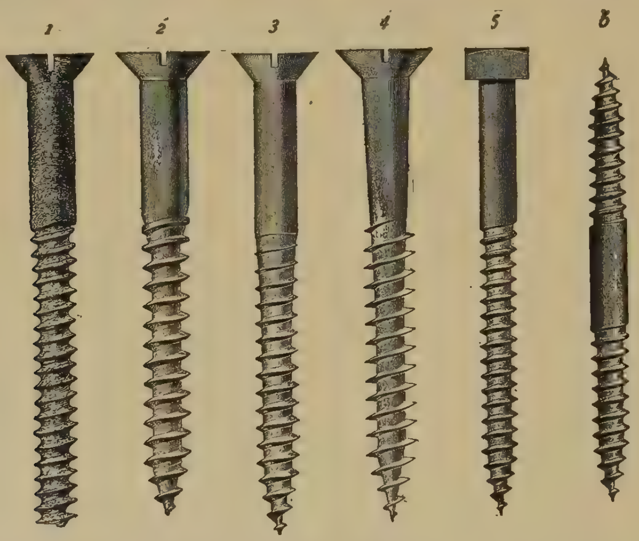
A common wood screw is a metal fastening used to attach either a piece of wood or metal to a timber into which the screw penetrates. As illustrated in Fig. 11*c* it contains a slotted head, a smooth shank, and a thread attached to a core which terminates in a point. The core is tapered so that its pressure on the sides of the hole increases as it is driven deeper.

Screws are used instead of nails when greater holding power is required, to avoid the danger of splitting or other injury, to secure better fitting, or to separate the parts readily when this may be desired afterward. Screws are classified according to their use, the shape of the head, the form of the thread, or according to some special feature of their construction. The flat head is used when the material is thick enough to allow the head to be countersunk in it; other forms are used either for thinner material or for ornamental purposes or both.

Figures 11*a-d* illustrate the progress made in the manufacture of wood screws. No. 1 is from an old sample card imported by Jonathan Congdon of Providence; No. 2 was invented by Thomas Sloan in 1846; and Nos. 3 and 4 are the invention of Charles D. Rogers in 1876 and 1892, respectively. In the forms of 1846 and 1876 the thread is cut on the body of the screw blank, but that of 1892 is produced by a cold forging process.

The advantages claimed for the form invented in 1892 are that it has a forged nick, a point that is exactly centered, and a metallic skin over the entire surface, making it stronger than a cut screw; that it requires only one size of hole to be bored, less power to drive it in, and has greater holding power. However, the cut screw of 1876 still remains in general commercial use, notwithstanding the decided superiority of the later design which is produced by cold forging.

Wood screws were in use long before 1760, when a patent was issued for cutting screws by machinery, having previously been made entirely by hand. In 1832 hand threaders were still in use in Providence, R. I., and the use of machinery for manufacturing screws was limited. In 1854 a greatly improved machine was invented in the United States which revolutionized screw-making. In about 1870 steel was first used for commercial wood screws, and by 1890, or very soon thereafter, the use of iron for this purpose was practically discontinued.



Figs. 11*a–f*.—Four Wood Screws, Coach Screw and Dowel Screw.

Screws are made in various lengths up to 6 in., and some lengths are made in as many as 18 different diameters. The gage numbers ranging from 0 to 30 refer to the American screw gage, which differs from the ordinary wire gage. The relation between the diameter of the shank and the screw gage is given by the equation

$$D = 0.0578 + 0.01316\,N,$$

in which D is the diameter in inches, and N the number of the screw gage. Screws are made of iron, steel, brass, copper, and other materials, with different kinds of finish or plating. In damp places brass screws should be used. Tables giving the available sizes may be found in the catalogs of screw manufacturers or of wholesale hardware dealers.

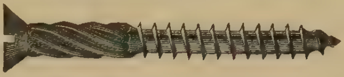
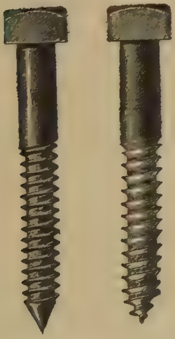


FIG. 11*g*.—Helicoid-shank Screw.

In good practice a hole is bored of the diameter of the core before inserting the screw. In hard wood a second hole is required of the diameter and depth of the shank to keep the wood from splitting or the screw from twisting off.

The helicoid-shank wood screw has been devised to avoid the necessity in hard wood and in fine work of boring the second hole for the shank of the common form of screw. In this improved type (Fig. 11*g*), the shank has a series of helical V-shaped ribs or threads of very large pitch which serve to ream out the hole to an exact fit during the operation of turning the screw.

When a common screw is partially driven into the wood with a hammer before using the screw-driver to drive it home, the fibers are seriously injured, and the holding power of the screw materially reduced. A cold-forged drive screw has been invented which, owing to its peculiar form of thread of large pitch, turns like a screw when driven with a hammer, and does not break the fibers of the wood. Its holding power equals that of a common screw, as frequently inserted, while its cost is less.


FIGS. 11*h* and *i*.—Lag and Coach Screws.

Lag screws and coach screws are large screws with square heads like that of a bolt and are turned with a wrench. They are used in heavy timber construction. Lag screws have cone points, as in Fig. 11*h*; coach screws have gimlet points, as in Figs. 11*e* and *i*. The latter illustration shows also the rachet thread which is sometimes used on either form of screw. A hole must first be bored of the same diameter and depth as the shank, and then continued with a diameter equal to that at the root of the thread. The diameters range from $\frac{1}{4}$ to 1 in., and the lengths from $1\frac{1}{2}$ to 12 in. See catalogs of the manufacturers.

The screw spike has come into use in this country since 1906 to fasten track rails to the cross-ties on elevated railroads and on ordinary ballasted track, in a more effective manner than by means of the common railroad spike which so seriously injures the fibers of the wood thus reducing its strength and promoting decay. In Arts. 7 and 8 references are given to articles and pamphlets in which their proportions and strength are given, and a history of their extensive use in Europe.

PROBLEM 11. By means of a screw manufacturer's catalog, prepare a table giving the lengths of lag screws for the different diameters.

ART. 12. HOLDING POWER OF COMMON SCREWS

An extensive investigation of the holding power of ordinary cut wood screws was made in the laboratory of the College of Civil Engineering, Cornell University, by N. M. Works and W. J. Graves in 1897 and 1898. The tests were made with white pine, yellow pine, and white oak, the screws being furnished by the American Screw Company.

The investigation included 10 lengths of screws from 1 to 6 in., and 8 diameters from No. 4 to No. 30, or 0.1105 to 0.4520 in., the diameter at the root of the thread ranging from 0.070 to 0.235 in. Three to six different diameters of screws were tested for each length, while from 3 to 5 specimens were tested for each size and each kind of timber, making 503 tests, in addition to several hundred preliminary ones to determine the proper relations of diameter of hole to that of screw, the effect of driving part way with a hammer, and other conditions.

It was found that except in soft wood and for very small sizes of screws the holding power is increased by boring a hole before inserting the screw. In white pine the best results were obtained by varying the diameter of the hole from 82 to 100 percent of the diameter of the inner cylinder or core of the screw, in yellow pine from 90 to 100, and in white oak from 85 to 105 percent. The ratio increases as the diameter of the screw decreases. It also increases with the hardness of the wood. Practically the same relations were obtained with the screws parallel to the fibers of the wood as when perpendicular to the fibers. The depth of the hole should extend to the section where the diameter of the threaded portion diminishes rapidly as it approaches the point.

Individual tests with holes of the same ratio to the core of the screw and in the same stick of timber give a variation in holding power far greater than that of the average holding power for different-sized holes between the limits given above. This fact indicates that lack of homogeneity in timber influences holding power far more than the given changes in the size of hole. It will therefore be sufficiently precise for most purposes to make the diameter of the hole equal to that of the core of the screw.

Tests for the proper size of hole for the shank of a screw indicate that in white pine its diameter should be about 77 percent of that of the shank, and in the harder woods about 83 percent. However, since the holding power due to the friction of the shank averages only about 5 percent of the total, and it is secured by developing a tendency to split the timber, it is usually best to bore the hole of the same diameter as the shank, and to a depth slightly less than that where the thread begins when the screw is in place.

In longitudinal sections of timbers after screws, inserted perpendicular to the fiber, are partly pulled out, inspection shows that the fibers are bent up near the hole and pulled apart and damaged for a distance of about ¾ in. each way in white pine, and 1 in. each way in the harder woods.

Although ordinary observation of a cross-section of the timber fails to detect any damage to the grain beyond the reach of the thread, if the spacing of the screws is too small their holding power is reduced. It is desirable therefore to space the larger screws not less than 1 in. in a direction across the grain, and not less than 2 in. along the grain. In hard woods the corresponding spacing should not be less than 1½ and 2½ in. respectively When the screws are inserted parallel to the fibers, the damage due to pulling them out is mostly local, and hence the smaller spacing given for each kind of timber will be sufficient.

The effect of lubrication, as demonstrated by tests in white pine, is a marked reduction in the work needed to insert them and a small increase in holding power. The increase is greater in the harder woods where lubrication becomes a necessity, especially with the larger screws. The best lubricant is the heaviest one which will adhere to the threads. Tar soap gave the best results in these tests.

Driving the screws part way with a hammer, as is frequently done in practice, always lessens the holding power, by damaging either the fibers or the screws.

The holding power for screws inserted both perpendicular to and parallel to the fibers is shown graphically in Figs. 12*a*, *b*, and *c*. Each dot or circle represents the average value for one size of screw, and the equation of each line gives the relation between the holding power and the net shearing area, called the thread area on the diagrams. This area is the product of the effective length of the threaded portion and the outer circumference at the middle of the effective length. The effective length of the threaded portion of the screw excludes from 3 turns for No. 4 to $1\frac{1}{2}$ turns for No. 28 to allow for the tapering end. The relation between holding power and shearing area is expressed by a parabola in each case, the curvature being less for the soft than for the hard woods. For screws with thread areas less than 2 sq. in., which includes nearly all screws less than 4 in. long, the relation may be expressed with sufficient precision by a straight line, making the holding power directly proportional to the shearing area. Including all sizes and lengths of screws tested, the average resistance in pounds per square inch of shearing surface or thread

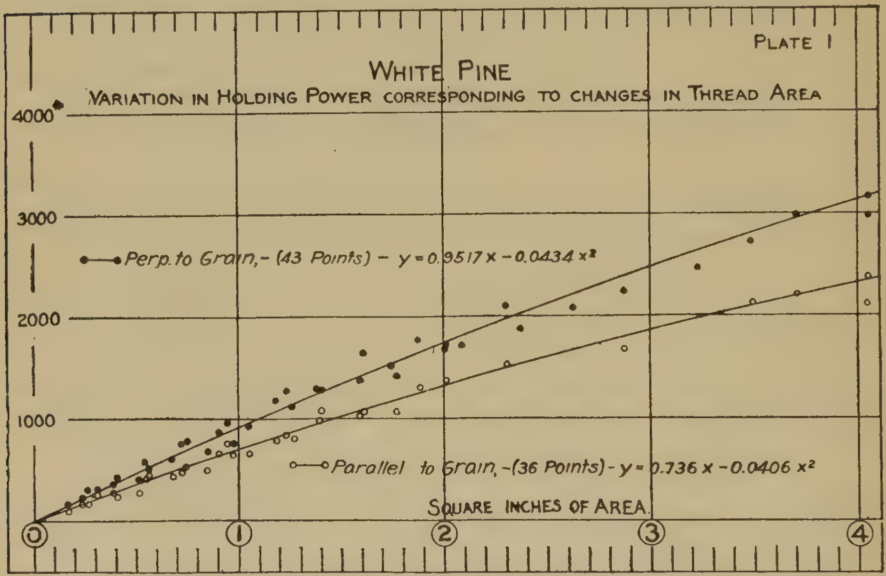


FIG. 12*a*.—Holding Power of Wood Screws in White Pine.

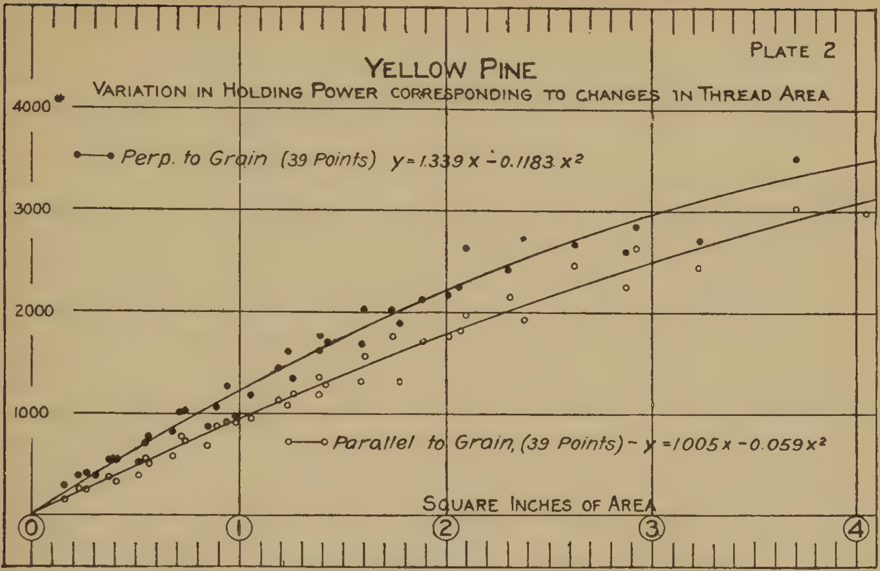


FIG. 12*b*.—Holding Power of Wood Screws in Yellow Pine.

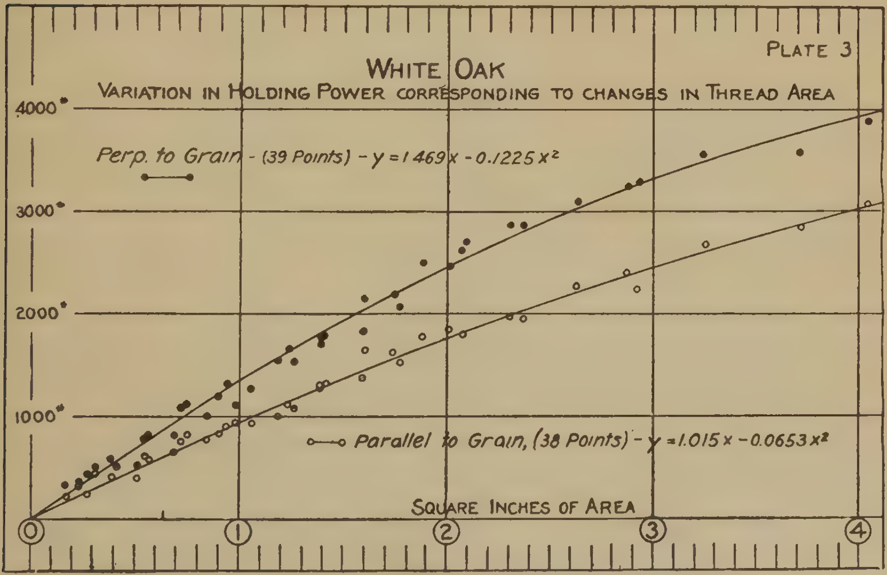


FIG. 12*c*.—Holding Power of Wood Screws in White Oak.

area is 905 in white pine, 1199 in yellow pine, and 1304 in white oak, the screws being inserted perpendicular to the fibers.

The average ratio of the holding power for screws parallel to the fibers, to that perpendicular to the fibers, was found to be 74 percent for white pine, 77 percent for yellow pine, and 78 percent for white oak. The average holding power in white pine is 72.5 percent of that in yellow pine and 69 percent of that in white oak, and the corresponding relation between yellow pine and white oak is 93 percent.

The holding power per pound is greater for the smaller sizes of screws. For screws of a given length it decreases as the diameters increase, and for those of a given diameter there is a slight increase with the length. The proportions of screws are an important factor in their holding power and depend upon the quality of the timber in which they are used. In long screws with the smaller diameters the heads pull off before the fibers of the wood fail, if hard wood is used, which indicates that such sizes should be employed in softer wood. For the kinds of timbers used in these tests the economical limit of length is about 4 in., and with this length the economical use of lag screws begins; for it is exceedingly difficult to insert longer screws with slotted heads into hard wood without damaging the screw. No. 12 screws from 2 to $2\frac{1}{2}$ in. long are the most economical sizes, so far as direct holding power alone is concerned.

A small crack or season check in the timber near a screw does not appreciably reduce the holding power, but a knot greatly increases it. There is practically no loss in holding power, due to removing a screw and reinserting it. In yellow pine the holding power of screws in the sap wood is less than in heart wood, the difference being 30 percent or more. Since the elastic limit of timber is about 75 percent of its ultimate strength, a relatively low ratio between the ultimate and safe holding power for screws may be adopted provided the number of screws used in the connection is not too small.

An approximate value of the holding power may be conveniently computed by means of the following table, which gives the ultimate holding power per linear inch of the total length of screw, expressed in pounds, upon the assumption that the entire threaded portion is engaged. For example, the holding power of a No. 12 screw 2 in. long in white pine is $2 \times 395 = 790$ lb., and of a

No. 20 screw 3 in. long in yellow pine is 3 × 618 = 1854 lb. In order to determine the safe strength, these values must be divided by a suitable factor.

Ultimate Holding Power of Wood Screws per Inch of Total Length When Inserted Perpendicular to the Fiber

Gage number	4	8	12	16	20	24	28	30
White pine	200	274	395	427	481	557	591	652
Yellow pine	295	400	505	547	618	675	732	
White oak	315	410	533	599	686	777	780	

The holding power of a drive screw equals about two-thirds of that of the common screw. The drive screw fails by overcoming the frictional resistance to turning. Lubrication greatly lessens its holding power, but is unnecessary, as the drive screw may be driven with ease into the hardest wood. With reference to holding power it is intermediate between the nail and the ordinary screw.

Another series of investigations was made at the Watertown Arsenal, including a little over 200 tests of wood screws, of Nos. 12 to 18, and ranging in length from $1\frac{1}{2}$ to $3\frac{1}{2}$ in. The average resistance in pounds per square inch was found to be 922 in white pine, 1493 in yellow pine, 2598 in white oak, and 2338 in California laurel. Holes were bored of a diameter equal to or slightly smaller than that at the root of the thread in the hard woods, but no holes were bored in the white pine. The average resistances in white and yellow pine with the screws inserted parallel to the fibers and no holes bored are 71 and 81 percent, respectively, of the corresponding values given in the preceding sentence. Since the same pieces of white and yellow pine were used as for the tests of nails, the corresponding resistances may be compared directly, showing that wood screws are about 2.3 times as effective in direct holding power as cut nails. For the detailed record of tests see Tests of Metals, etc., 1884, page 448.

Problem 12. A No. 24 wood screw, having a total length of 4 in., has an effective threaded length of 2.48 in. The outer and inner diameters of the threaded part are 0.368 and 0.253 in., respectively, and there are 7 threads per linear inch. Compute the resistance to withdrawal for an allowable compression of 390 lb. per sq. in. on the sides of the fibers of the wood into which it is inserted, on the assumption that the compressive area for a thread equals its horizontal projection.

ART. 13. HOLDING POWER OF LAG SCREWS

The holding power of lag screws was investigated by W. M. Macphail and T. T. Irving, and the results were published in Transactions Canadian Society of Civil Engineers, 1899, vol. 13, page 141. The screws were $\frac{3}{8}$, $\frac{1}{2}$, $\frac{5}{8}$, and $\frac{3}{4}$ in. in diameter; and Canadian red pine (Norway pine) and white oak were used as representatives of the soft and hard woods, respectively, the total number of tests being 317.

Two sizes of holes were bored for each diameter of screw, one equal to the diameter of the screw at the root of the thread and the other one-sixteenth inch larger. In the pine the holding power for the larger holes averaged 95 percent of that for the smaller holes, whereas in the oak the corresponding relation was 105 percent. It appears that the insertion of the threads in the hard wood caused more injury to the fibers in the case of the smaller hole, and thus reduced the resistance of the wood when the screw was pulled out.

By splitting the block and examining the wood around the screw it was found that when the hole was bored too small, the fibers were crushed but when it was of the right size, the thread in the wood looked clean-cut, the fibers were pressed, and the fit of the wood on the screw resembled the appearance of a nut on a bolt.

The holding power was also found to vary directly as the diameter of the screw, which is the same relation as that given in Art. 12 for smaller sizes of wood screws. When the screws were inserted parallel to the fibers, the holding power was about 108 percent of that when inserted perpendicular to the fiber in the case of pine, and about 95 percent in the case of oak.

The holding power in the pine was found to vary directly as the depth of penetration of the threaded portion of the screw. In no case was the screw driven deeper than the threaded length. In the case of oak, however, this relation was not so close. For screws driven 4½ in. the holding power per linear inch was about 8 percent less than when driven only 3 in. Since the screws were stressed nearly or quite to their ultimate strength, as several of them actually ruptured before the wood yielded, the unequal elongation of the screws apparently caused an unequal distribution of the pressure on the threads. For safe loads the elongation was much less, and the distribution may be regarded as uniform for practical purposes. The average ultimate holding power in the red pine was found to be about 780 and in the white oak about 1140 lb. per sq. in.

In the report entitled Tests of Metals, etc., at the Watertown Arsenal for 1902, page 563, are given the results of 252 tests of the holding power of lag screws, four of them in yellow pine, whereas the rest are about equally divided between Douglas fir and white oak. The diameters of the screws ranged from ½ to 1 in. Three sizes of holes were bored for each diameter of screw, one equal to the diameter of the screw at the root of the thread, and the others respectively $\frac{1}{16}$ and ⅛ in. smaller. In white oak there was practically no increased resistance due to the smaller holes, but in Douglas fir the average increase in resistance for the smallest holes was 5.2 percent over that for the largest, which equals the diameter at the root of the thread. Therefore, considering the greater effort required to insert the screw into the smaller hole, it is best to have the hole the same diameter as the core of the screw. It was also found that there is practically no difference between the average resistance for the common form of screw thread and the ratchet thread.

In this point the resisting area is computed as the area of a cylinder with a diameter equal to that of the outside of the thread as actually measured, instead of the nominal diameter, and a length equal to that of the undiminished threaded portion of the screw, thus excluding the tapering end. The average ultimate resistance in pounds per square inch is given in the following table:

Resistance of Lag Screws

(Expressed in pounds per square inch)

Diameter of screw, in.	½	⅝	¾	⅞	1
Diameter of hole, in.	⅜	½	$\frac{9}{16}$	⅝	$\frac{13}{16}$
Resistance in Douglas fir	869	657	689	882	593
Resistance in white oak	1603	1293	1232	1137	1175

The average resistance for all sizes of lag screws in Douglas fir is 739 and in white oak 1287 lb. per sq. in. The corresponding value for the four tests in yellow pine is 777 lb. per sq. in.

The average ultimate resistance per linear inch of penetration of thread is as follows:

RESISTANCE OF LAG SCREWS

(Expressed in pounds per linear inch)

Diameter of screw, in.	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1
Diameter of hole, in.	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{13}{16}$
Resistance in Douglas fir	1258	1348	1561	2291	1836
Resistance in white oak	2377	2484	2919	3185	3612

The corresponding resistance for the four tests of $\frac{3}{4}$-in. lag screws in yellow pine is 1830 lb.

Since the average holding power in white oak expressed in pounds per square inch is 74 percent greater than in Douglas fir, it appears that the resistance should be considered as proportional to the corresponding safe unit stress in compression on the side of the fiber. This relation is sufficiently precise to find the corresponding holding power for lag screws in other kinds of wood.

In the Watertown tests the resistance was also observed after the lag screws were withdrawn $\frac{1}{8}$, $\frac{1}{4}$, and $\frac{1}{2}$ in., respectively, whereas for a part of the tests the corresponding distances were $\frac{1}{8}$, $\frac{3}{8}$, and $\frac{7}{8}$ in. The resistances during withdrawal are given in the following table expressed as percentages of the ultimate holding power:

Distance withdrawn, in.	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{7}{8}$
Resistance, percent	92	76	55	44	14

These results show that even in cases of failure the reduction in the holding power of lag screws is gradual.

A small number of tests was previously made at the Arsenal. The average resistance for 26 tests in white pine is 781, for 7 tests in yellow pine is 966, and 4 tests in white oak is 1232 in. per sq. in. In white pine the sizes of screws tested are $\frac{3}{8}$, $\frac{1}{2}$, and 1 in. diameter, whereas in the other two kinds of wood only the 1-in. size was used. The record showed half-tone illustrations of the effect on the fibers of white pine and white oak, respectively, of pulling out 1-in. lag screws with the testing machine.

In another series of tests, with diameters of $\frac{1}{2}$ and $\frac{5}{8}$ in., the resistance in pounds per square inch was 991 in yellow pine, 613 in white cedar, and 735 in hemlock, the last value being an average of two tests.

A few tests made by P. Lobben were published in the American Machinist, vol. 18, page 964, Dec. 5, 1895. The sizes ranged from $\frac{7}{8}$ to $\frac{1}{4}$ in. in diameter; 7 tests in spruce gave an average ultimate resistance of 735 lb. per sq. in., and 1 test each in chestnut and pitch pine gave 691 and 783 lb. per sq. in., respectively.

Some experiments made at Sand Beach, Mich., gave the holding power of a 1-in. screw in white pine, with a $\frac{7}{8}$-in. hole bored, as 435 lb. per sq. in. This value is the average for 12 tests in two sticks of timber. When the holes were bored either $\frac{13}{16}$ or $\frac{15}{16}$ in., the resistance was smaller.

PROBLEM 13. Refer to Tests of Metals for 1902, and compute the average percentage of increased holding power of each size of lag screw in Douglas fir due to boring the holes smaller than that of the screw at the root of the thread.

ART. 14. LATERAL RESISTANCE OF LAG SCREWS

The accompanying table gives the results of tests on fourteen joints fastened by lag screws as reported by H. D. Dewell in Engineering News, vol. 76, pages 162 and 796, July 27 and Oct. 26, 1916. The timber used was Douglas fir, four of the tests being on joints in which a $\frac{1}{2}$-in. steel plate was fastened to a timber block, whereas the other tests were on joints in which wooden plates, $1\frac{1}{4}$ to 2 in. thick, were lagged in symmetrical pairs to an 8 × 8-in. block. All plates were held with three lag screws each.

Joint Letter or Number	Size of Lag, In.	Thickness of Side Pieces	Total Load at Slip of $\frac{1}{16}$ In., Lb.	Load per Lag at Slip of $\frac{1}{16}$ In., Lb.	Total Final Load, Lb.	Final Load per Lag, Lb.	Final Slip, In.
7	$\frac{3}{4} \times 4\frac{1}{2}$	$\frac{1}{2}$-in. steel plate	7,700	2570	20,000	6,670	0.92
8	$\frac{3}{4} \times 4\frac{1}{2}$	$\frac{1}{2}$-in. steel plate	6,700	2230	19,700	6,570	0.75
9	$\frac{7}{8} \times 5$	$\frac{1}{2}$-in. steel plate	6,700	2230	30,000	10,000	1.14
17	$\frac{7}{8} \times 5$	$\frac{1}{2}$-in. steel plate	7,500	2500	30,000	10,000	1.15
A	$\frac{7}{8} \times 5\frac{1}{2}$	2 in.	12,500	2080	49,100	8,180	1.40
B*	$\frac{7}{8} \times 5\frac{1}{2}$	2 in.	14,400	2400	34,000	5,670	1.61
C	$\frac{3}{4} \times 4\frac{1}{2}$	$1\frac{1}{4}$ in.	12,400	2070	34,000	5,670	1.49
D	$\frac{7}{8} \times 5\frac{1}{2}$	2 in.	10,700	1780	44,400	7,400	1.69
E*	$\frac{7}{8} \times 5\frac{1}{2}$	2 in.	10,400	1730	33,000	5,500	1.70
F	$\frac{3}{4} \times 4\frac{1}{2}$	$1\frac{1}{2}$ in.	12,000	2000	33,400	5,570	1.14
G	$\frac{7}{8} \times 5\frac{1}{2}$	2 in.	13,800	2300	44,700	7,450	1.44
H	$\frac{3}{4} \times 4\frac{1}{2}$	$1\frac{3}{8}$ in.	9,400	1570	28,885	4,820	1.30

* Lag screws had bearing across the grain of the center timber. All other bearings were along the grain.

In Art. 6 it is stated that for nailed joints the elastic limit comes at a slip of about $\frac{1}{16}$ in. Hence, using a factor of safety of two, the safe load per lag screw for steel plate fastenings is 1200 lb. It will be noted that for a slip of $\frac{1}{16}$ in. the $\frac{3}{4}$-in. lag screw is as strong as the $\frac{7}{8}$-in. ones, but for ultimate loads the latter are considerably stronger.

In Art. 8 a formula is developed for determining the safe resistance of a track spike to lateral displacement. Applying this formula to the steel-plate joint and assuming the diameter of the lag screw to be constant and equal to the nominal diameter of the lag screw and using the limiting bearing stress of 1300 lb. per sq. in., there is found for the $\frac{3}{4}$- by $4\frac{1}{2}$-in. lag screw and $\frac{1}{2}$-in. plate a safe resistance to lateral displacement of 770 lb. For the $\frac{7}{8}$- by $5\frac{1}{2}$-in. lag screw the value is 1040 lb. The maximum flexural stresses are respectively 10,700 and 13,400 lb. per sq. in.

For slips of $\frac{1}{16}$ in. the joints in which the lag screws bore across the fibers of the timber in the main block carried as much load as where end bearing existed. However, the maximum loads were much less.

In the articles referred to at the beginning of this article Dewell recommends the following allowable loads for timber in a dry condition:

Type of Joint	Pounds per Screw
Metal plate lagged to timber, $\frac{3}{4}$- by $4\frac{1}{2}$-in. lag screws	1030
Metal plate lagged to timber, $\frac{7}{8}$- by 5-in. lag screws	1200
Timber planking lagged to timber, $\frac{3}{4}$- by $4\frac{1}{2}$-in. lag screws	900
Timber planking lagged to timber, $\frac{7}{8}$- by 5-in. lag screws	1050

Art. 15. Dowels

A dowel is an iron or wooden pin extending into, but not through, two members of a structure to connect them. The dowel is usually placed perpendicular to the surface of contact. It is most frequently employed to connect the end of one timber to the side of another, to prevent the lateral displacement of the former. The dowel differs from a drift bolt in general by being larger in diameter and shorter in length. It has neither point nor head, and is used in the same form as cut from the rods, provided it is straight.

In Fig. 15*a* the dowel holds the tip of the follower pile in place on the butt of the lower pile. In Fig. 15*b* two dowels are used in the same dowel joint, and Fig. 15*c* shows the hip joint of a Howe bridge truss, which contains two dowels and one boat spike. The upper dowel is 1 in. in diameter and 8 in. long. See also the dowel joints in the Howe truss of an exposition building illustrated in Fig. 81*a*.

In a report on framing timber trestle bridges, a Committee of the American Association of Railway Superintendents of Bridges and Buildings made the following reference to dowels: "The second method of holding together trestle joints is by means of dowels. . . . This method, although cheap as to first cost, has many objectionable features. First, it does not give a rigid joint, and in renewals it is seriously in the way of renewing defective timber where the time between trains is limited, for whereas a tenon can be readily cut off with a saw, the dowel would have to be cut off with a heavy chisel bar and a maul, or the wood cut away from it sufficiently to release the timber."

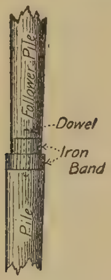


Fig. 15*a*.

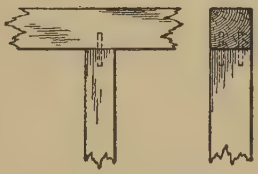


Fig. 15*b*.—Dowel Joint.

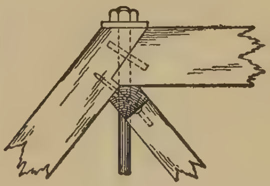


Fig. 15*c*.

In trestles the sizes of dowels usually range from 1 to 2 in. in diameter and from 6 to 12 in. in length, but occasionally these limits are passed. Holes are bored slightly less in diameter than the dowels, so that they will fit closely after the timbers are firmly driven together. The dowels and the whole joint surface should be thoroughly painted or covered with some preparation of coal tar, carbolineum, or other preservative before they are put together, thus making the joint practically watertight and preventing the early decay of the timber. In this respect the dowel joint is superior to the mortise and tenon joint, since with the class of timber used in trestles it is impracticable to make the latter joint tight for any length of time.

The dowel, however, does not make as rigid a joint as the mortise and tenon, and some difficulty is experienced in erecting large trestle bents framed in this manner. After the bent is in place, however, and the diagonal and longitudinal braces attached, there is practically no deficiency in rigidity. The time and expense for framing with dowels are much less than with the mortise and tenon, and a much larger surface of contact is available for bearing in timbers of equal size.

It is customary to extend the dowel equal distances into the two timbers joined, but good design requires a longer penetration into the end of one timber than into the side of the other, since there is a greater tendency to split the former.

Additional examples of the use of dowels in framing may be seen in Plate III, in the connection of the floor beams to the foundation posts, and in a roof truss in Art. 72. Figure 15*d* shows an intermediate joint of a two-story trestle in which the braces are compression members.

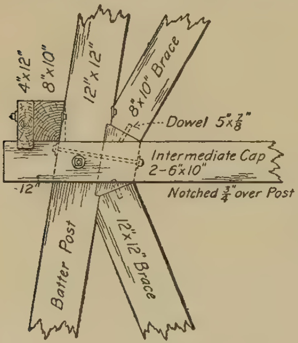


FIG. 15*d*.

Dowels are also employed in trestle construction to anchor the cross-tie to the stringers when the stringers are covered with galvanized iron in order to prolong their life. The size used for this purpose is $\frac{3}{4}$ in. in diameter and 5 in. long, the holes being bored $\frac{11}{16}$ in. in diameter to insure a driving fit. See an article on Protection of Members in Wooden Bridge Structures, Engineering Record, vol. 59, page 637, May 15, 1909. A dowel screw is shown in Fig. 11*f*.

PROBLEM 15. Prepare a list of the illustrations in Chapters IV and V in which dowels are shown.

ART. 16. WOODEN PINS AND TREENAILS

Wooden pins are round pieces of hard wood inserted through the timbers of a joint to hold them together. A common example of its use is that in which the pin prevents a tenon from drawing out of a mortise (see Art. 34).

Treenails are slender pieces of hard wood used in a manner similar to the use of iron nails to fasten boards or timbers together. They are employed where iron nails or spikes would cause injury by rusting, and where copper nails are too expensive. Treenails are slightly tapered (Fig. 34*d*) to facilitate driving and are made of different diameters and lengths according to the sizes of the timbers which they unite. Sometimes treenails are used as pins in bringing the surfaces of joints firmly to their bearings and retaining the timbers in place.

An illustration of the use of treenails is given in Fig. 16*a*, which shows the detail of a portion of the wooden curbing for the concrete pivot pier foundation of the Charlestown bridge at Boston. The curbing has a mean diameter of 75 ft. and is 32 ft. high. Each course consists of 24 spruce planks 3 by 12 in., each plank being about 10 ft. long. The planks were planed on one side to an even thickness, sawed to proper length and level, and spiked and treenailed together. The treenails are made of oak and are 1 in. in diameter.

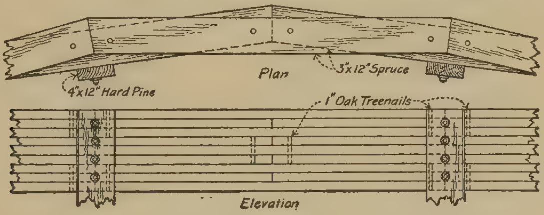


Fig. 16*a*.—Wooden Curbing for Pier Foundation.

Some small-size shearing tests across the grain on specimens $\frac{5}{8}$ in. in diameter were made by John C. Trautwine. Two tests in double shear were made for pins of 24 different kinds of wood. Some of the largest values expressed in pounds per square inch are: ash, 6280; beech, 5223; birch, 5595; dogwood, 6510; gum, 5890; hickory, 6045 to 7285; locust, 7176; maple, 6355; white oak, 4425; live oak, 8480; and southern yellow pine, 5735.

In a table given in Manual of Railroad Engineers, by George L. Vose, it is shown that the resistance of seasoned oak treenails to shearing across the fiber is greater when they connect 6-in. planks than 3-in. planks. Apparently the treenails are not allowed to bend as much in the thicker planks before they rupture. For diameters of 1, $1\frac{1}{4}$, $1\frac{1}{2}$, and $1\frac{3}{4}$ in. the ultimate shearing stresses in pounds per square inch were found to be 4282, 4130, 3435, and 3203 when in the 3-in. planks, and 4538, 4130, 3910, and 3967 when in the 6-in. planks.

The species most frequently used are oak, locust, and hickory. In designing wooden pins and treenails it is necessary to consider the bearing on the side of the fiber for the pin as well as the bearing either on the side or end of the fibers for the wood in which the hole is bored, depending upon the direction in which the pressure is transmitted. In construction it should be remembered that the resistance to bearing on the side of the pin is greater when the line of pressure is perpendicular to its annual rings.

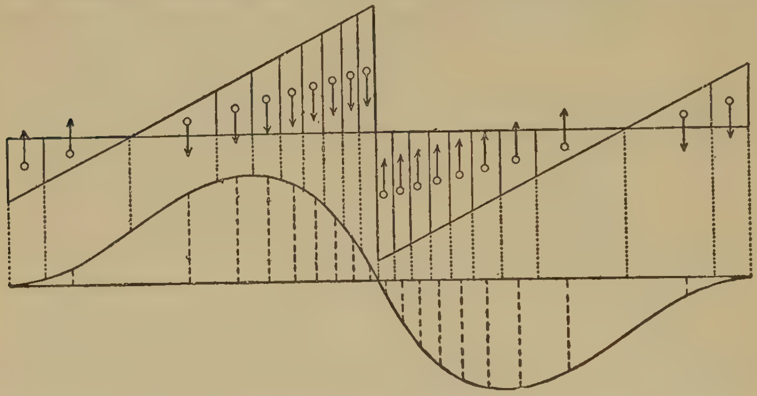


FIG. 16*b*.—Bending Moment Diagram for a Wooden Pin.

For an example, in the investigation of the strength of a pin, let oak pins 2 in. in diameter connect two planks each $2\frac{7}{8}$ in. thick which tend to slide past each other longitudinally. The distribution of the pressure or loading on the pin in each plank is like that of a railroad spike as illustrated in Fig. 8*b*, provided the distance e equals zero. P therefore indicates the magnitude of the vertical shear in one pin in the section between the two planks. Formulas (1) in Art. 8 give values of $c = \frac{1}{3}l$ and $a = \frac{2}{3}l$ when $e = 0$, the distance l being the length of the pin in one plank. The loading of the pin in both planks is indicated in Fig. 16*b*, and the same diagram also gives the bending moment diagram, which was constructed graphically. The load areas were so subdivided as to make each one represent a load of $\frac{1}{6}$ P. The pole distance was taken equal to $\frac{1}{2}P$, and the maximum moment ordinate was found to measure 0.85 in. The maximum moment is therefore $0.85\ P/2 = 0.425\ P$ in.-lb., P being expressed in pounds.

The section modulus of a 2-in. pin is 0.7854 in.^{3}, and for a unit stress in the outer fiber of 1500 lb. per sq. in., the resisting moment is 1178 in.-lb. Equating the resisting moment to the bending moment, the value of the shear P is found to be 2770 lb. For a unit stress in shear across the fibers of the pin of 1000 lb. per sq. in. the value of P is 3142 lb.

Assuming that the bearing of a round pin of oak is equivalent to 0.74 times the bearing on a square pin, and that the allowable compression on the side of the fiber is 700 lb. per sq. in., the maximum bearing on the pin per linear inch is $0.74 \times 2 \times 700 = 1036$ lb. The load $4\ P/3$ is distributed over a distance of 1.92 in., but the maximum load is twice the average load, hence $4\ P/3 = (1036/2)\ 1.92$, whence $P = 995$ lb. This computation shows that the compression on the side of the fibers of the pin really determines the magnitude of the shear between the planks which it can resist. The use of hollow metal pins, cut from pipe, would materially increase the strength of the joint.

Problem 16. If a pin made of extra hydraulic tubing with an inside diameter of 1.49 in. and a thickness of 0.20 in. is substituted for the oak pin in the example given in this article, compute the shear between the plank which it will resist.

Art. 17. Wooden Keys and Wedges

Wooden keys are inserted in some types of joints to bring the surfaces into close contact or to prevent some piece from being withdrawn. Illustrations of their use are given in Figs. 31*e*, *k*, *m*, *n*, 34*k*, and 35*k*.

When a key is used to prevent two adjacent timbers from sliding longitudinally with respect to each other, it is best to design the key with the fibers parallel to those of the timbers in which it is inserted, as indicated in Fig. 17*a*. The longitudinal pressure of the timbers is taken directly by the ends of its fibers, while the key is subject to shear parallel to its fibers, and to rotation which is resisted by bearing on its sides. The compression on the half of each upper or lower side of the key is distributed as the ordinates to a straight line, the maximum pressure being at the end. The tendency of the key to rotate will cause the timbers to separate unless they are held together by one or more bolts. The tension in the bolt is therefore directly related to the pressure on the sides of the key.

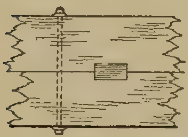


FIG. 17a.—Wooden Key.

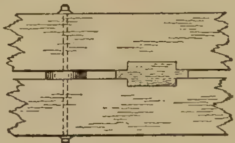


FIG. 17b.—Key with Shoulders.

For example, let it be required to design a rectangular wooden key of Western hemlock to resist a longitudinal shear of 10,000 lb. between two timbers 6 in. wide. The specified unit stresses, expressed in pounds per square inch, are: Shear parallel to the fiber, 240; compression on the ends of the fiber, 1800; and compression on the side of the fiber, 330. Placing the fibers of the key parallel to the timbers, the length required to resist shearing is $10{,}000/(6 \times 240) = 6.945$ or 7 in. The required depth of the key is $2 \times 10{,}000/(6 \times 1800) = 1.852$ or $1\frac{7}{8}$ in., which is preferably increased to 2 in., so as to cut the keys from 2-in. plank.

The moment of rotation due to the eccentric compression (Fig. 17c) at the ends of the key is $10{,}000 \times 2/2 = 10{,}000$ in.-lb. To find the length required to keep the allowable pressure on the sides of the fibers within the allowable limit of 330 lb. per sq. in., let the length be designated by l. The total compression on each side of the key is $(330/2)\ (6l/2) = 495l$ lb., and the lever arm of the resultant couple is $2l/3$, making the moment $330l^2$. Equating the two moments, there is obtained $l = 5.505$ or $5\frac{5}{8}$ in. As this value is less than that required for shear, the latter will govern, making the length to be adopted 7 in.

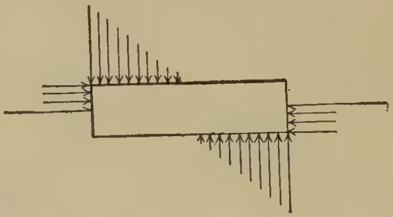
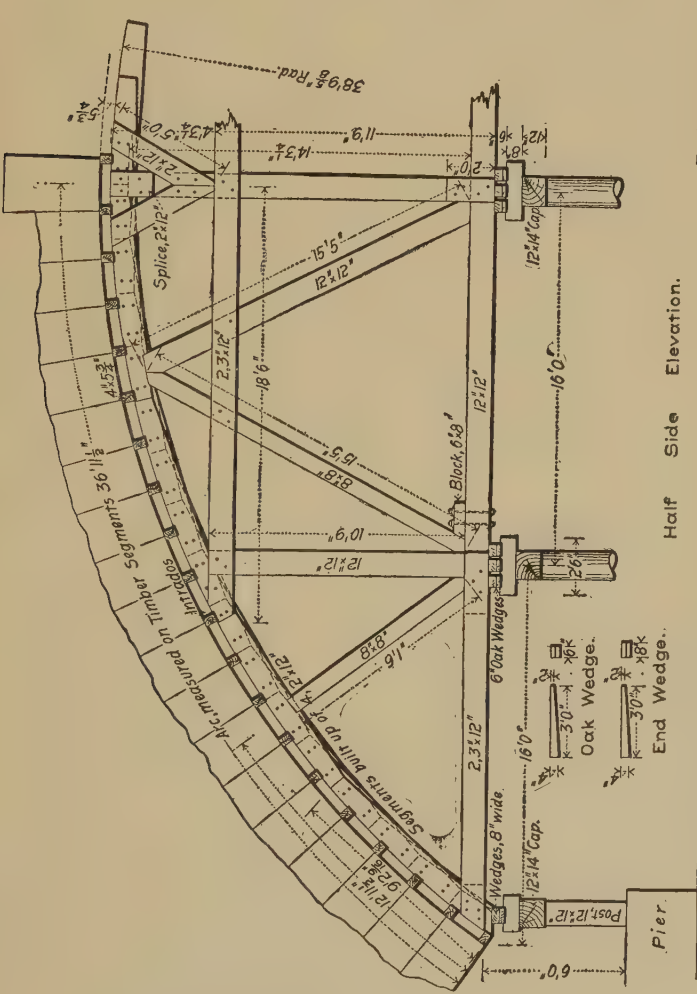


FIG. 17c.—Compression on a Key.

Wedges are extensively used under falsework upon which arches and other bridges are erected, so that when the structure is ready to become self-sustaining the support of the falsework may be gradually released by slowly withdrawing the wedges. (See Fig. 17d.)

Large wedges are usually made of oak wood because of its large resistance to compression on the side of the fiber. For the same reason small wedges are frequently made of hickory, this wood being also very tough. For large wedges under heavy loads a taper of 1 in 10 may ordinarily be used or in extreme cases 1 in


38'9⅝" Rad.
Splice, 2"x12"
2"x12"
15'5"
12"x12"
2,3"x12"
18'6"
15'5"
8"x8"
Block, 6"x8"
12"x12"
10'9"
12"x12"
8"x8"
9'1"
2,3"x12"
Arc measured on Timber Segments 36'11½"
Intrados
Segments built up of
12"x14" Cap
16'0"
6" Oak Wedges
Wedges, 8" wide
12"x14" Cap
Post, 12"x12"
Oak Wedge
End Wedge
Pier
6'0"
Half Side Elevation.

Engineering News.

FIG. 17*d*.—Centering for Stone Arch Bridge of the Chicago, Milwaukee and St. Paul Railway over Rock River at Watertown, Wis. (See Fig. 80*i*.)

20. Under light loads the taper is often increased to about 1 in 6, or 2 in. per foot. In order to compute the static force necessary to drive up or to release wedges, it is necessary to know the friction between surfaces of wood under pressure.

The friction of wood upon wood at high pressure was determined experimentally by E. H. Messiter and R. C. Hanson in 1894, and an abstract of the results was published in Engineering News, vol. 33, page 322, May 16, 1895. Eighty-one tests were made at pressures ranging from 100 to 1000 lb. per sq. in., chiefly on yellow pine. When the sides of the fibers are placed longitudinally on each side of the rubbing surface, the coefficient of friction is 0.365 when the surfaces are rough as they come from the lumber yard, and 0.215 when the timbers are planed smooth. For smooth or sandpapered surfaces, the coefficient of friction is 0.227 when the ends of fibers rub against the sides of fibers, and 0.323 when the ends of fibers rub against the ends of fibers. When the fibers are inclined on each side of the rubbing surface, either in the same or in opposite directions, the coefficient ranges from 0.255 to 0.315, the larger value being for an inclination of 30 deg., and the smaller one for 45 deg. The mean value for 64 tests on yellow pine was found to be 0.290. In every case the surfaces were found to be polished after the completion of the test; and in the case of very pitchy specimens the specimens were found to be sticky, especially after tests at very high pressures. A small number of tests was made also on fairly dry spruce which gives a mean value of 0.422 as the coefficient of friction.

In striking the centers of the Piney Branch arch bridge at Washington, D.C., the coefficient of friction was observed by W. J. Douglas, its value being about 0.4. The wedges had been greased, but it was thought that the grease did not facilitate the striking. The maximum pressure on the large wedges at the center of the span was 280 lb. per sq. in., whereas that on the other wedges varied from 160 to 200 lb. per sq. in. See Art. 80 and Engineering News, vol. 57, page 682, June 20, 1907.

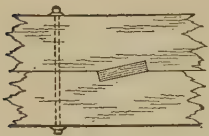


Fig. 17*e*.—Wooden Key.

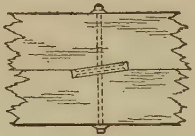


Fig. 17*f*.—Cast-iron Key.

By inclining the keys as in Fig. 17*e* the shear parallel to the fibers is avoided, and the compression at each end is distributed over its full depth. Figure 17*g* shows the use of keys placed in this position in the construction of double-leaf gates for the Lockport locks of the Chicago Drainage Canal. Each leaf is built of single horizontal courses of Douglas fir fastened together by white oak keys and vertical through bolts. The weight of the leaf tends to sag its movable end and to cause the courses to slide on each other.

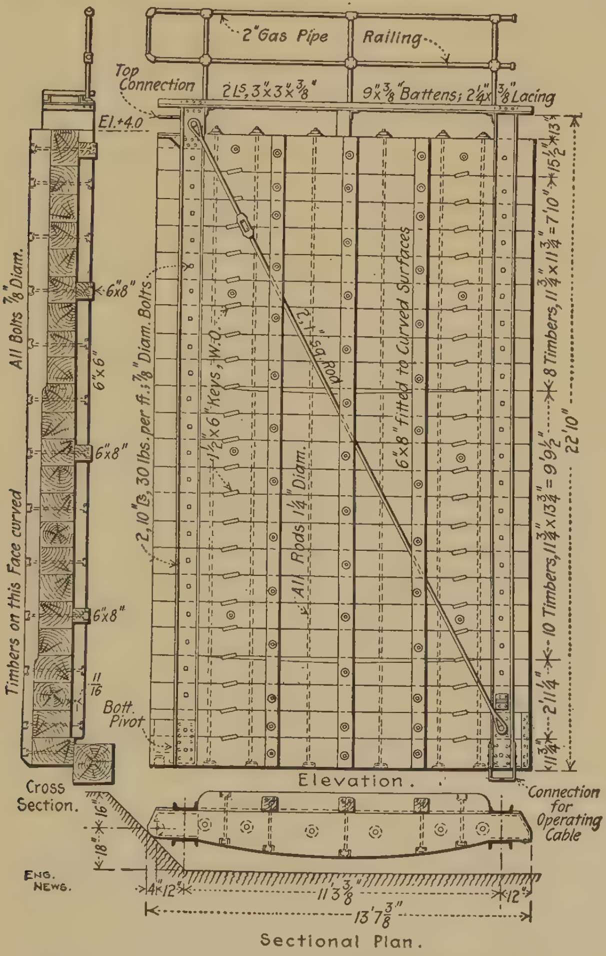

2" Gas Pipe
Railing
Top Connection
2 Ls, 3"x3"x3/8"
9"x3/8" Battens; 2 1/4"x3/8" Lacing
El.+4.0
All Bolts 7/8" Diam.
Timbers on this Face curved
6"x8"
6"x6"
6"x8"
6"x8"
11/16
Bott. Pivot
2, 10" [, 30 lbs. per ft.; 7/8" Diam. Bolts
1 1/2"x6" Keys, W.O.
All Rods 1 1/4" Diam.
1 1/4" Diam. Rod
6"x8" Fitted to Curved Surfaces
13"
15 1/2"
8 Timbers, 11 3/4"x11 3/4" = 7'10"
10 Timbers, 11 3/4"x13 3/4" = 9'9 1/2"
2'11 1/4"
11 3/4"
22'10"
Elevation.
Cross Section.
Connection for Operating Cable
16"
18"
Eng. News.
4"
12"
11'3 3/8"
12"
13'7 3/8"
Sectional Plan.

FIG. 17*g*.—Lock Gates on Chicago Drainage Canal.

This is resisted by the keys and by the diagonal rods which are drawn tight by means of turnbuckles. See Engineering News, vol. 60, page 512, Nov. 12, 1908.

Round keys of white oak 2 in. in diameter were extensively used in the temporary wooden building construction at the St.

Louis Exposition (Art. 29). They were employed instead of rectangular keys in fish-plate splices in columns and truss chords and between the main timbers forming composite columns. For an illustration of their use in similar construction at the Pan-American Exposition see Figs. 53*c* and 79*c*.

Problem 17. Compute the total force required to release the large wedges of the Piney Branch arch centers.

Art. 18. Anchor Bolts

The object of an anchor bolt is to fasten in place or hold down to the supporting masonry some part of a structure, or machine. An anchor bolt has a thread cut at its upper end to engage a nut, and the lower end is embedded in the masonry. The bolt is embedded by drilling a hole in the masonry, setting the bolt in position, and filling the hole with melted lead or sulphur or a grout of neat cement. Since cement has come into such extensive use in construction, it has largely replaced other material for this especial purpose. Sometimes the anchor bolts are built into position as the stone or brick wall is constructed, and in this case the bolt has a nut at the lower end also, which acts against a bearing plate.

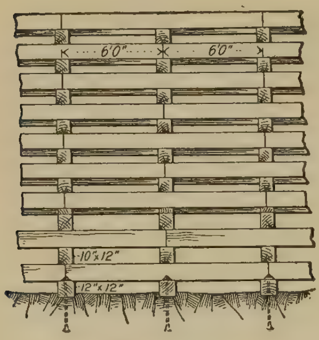


Fig. 18*a*.—Anchorage of Wooden Dam.

In some experiments made to determine the relative value of Babbitt metal, lead, and sulphur, in fastenings which were not to penetrate over 6 in. into stone, it was found that sulphur gave the most satisfactory results. In two tests with sulphur setting, the stone broke under a stress of 950 lb. per sq. in. of embedded surface of bolt.

In Engineering News, vol. 24, page 53, July 19, 1890, are given the results of 14 tests, divided equally between bolts $\frac{3}{4}$ and 1 in. in diameter, set in holes $1\frac{3}{8}$ and $1\frac{5}{8}$ in. in diameter drilled $3\frac{1}{2}$ ft. deep into a ledge of solid limestone. The ends of the rods were ragged. The cement setting was shown to be superior to that of lead and sulphur, both as to strength and ease of application. Four additional tests were then made with 1- and 2-in. bolts, set in stones 2 in. thick, two of the bolts being threaded and two plain. When the bolts were pulled very slowly, the first indication of yielding occurred at about 650 and 520 lb. per sq. in. of embedded surface for the 1- and 2-in. bolts respectively. On continuing the tests at greater speed, the stones finally split at considerably higher stresses. The threaded bolts indicated no superiority over the plain ones.

A few tests made by Robert Moore were reported in Engineering Record, vol. 23, page 209, Feb. 28, 1891. In one case a 2-in. plain rod set $11\frac{1}{2}$ in. deep with portland cement began to yield at a resistance of 470 lb. per sq. in., but the cement setting did not entirely part at 927 lb. per sq. in., when the stone broke. In another case with a threaded bolt the stone began to yield and finally broke when the resistances were 443 and 692 lb. per sq. in. respectively, without developing the full strength of the cement joint.

A larger number of experiments was made by E. S. Wheeler at St. Mary's Canal, and were recorded in Report of the Chief of Engineers, U. S. A., 1895, pages 2917 and 2940. The rods were plain, being $\frac{1}{2}$, 1 and $1\frac{1}{4}$ in. in diameter, both round and square. They were embedded about 8 to 10 in. in mortar composed of one part of portland cement and two parts of limestone screenings passing $\frac{3}{8}$-in. slits. The mean adhesive resistance for 3 tests of each size and form of rod was found to vary from 434 to 562 lb. per sq. in., the general mean for 6 sets being 511 lb. per sq. in. One-inch square twisted rods with 1 to 3 turns in 8 in. developed less than 8 percent greater resistance than the plain rods. Some additional tests were made by substituting ordinary river sand for the limestone screenings, giving an average adhesive resistance of 264 lb. per sq. in. On using four parts of sand instead of two, the adhesion was reduced more than half.

A series of tests was made in 1905 by Edward Holmes in the civil engineering laboratory of Cornell University. The bolts were of $\frac{7}{8}$-in. wrought iron, 18 in. long, with split bushing at the lower end. They were set 6 in. deep into 10-in. cubes of shale rock known locally among stone cutters at Ithaca marble. The holes were cut 2 in. in diameter. Cement grout, sulphur, and lead were used respectively in setting 5 bolts each. The cement was allowed to set 14 days before testing, except one which was tested after 8 days. The bolts set in sulphur or lead were tested after 24 hours.

The average results of the tests are as follows: In cement the bolts showed signs of loosening at 16,340 lb., and pulled out at 21,640 lb.; for the setting in sulphur the corresponding values are 13,140 and 18,480 lb.; and in lead, 8000 and 10,980 lb. The minimum resistances at first signs of loosening are 12,000 lb. for the cement tested after 8 days only, 7600 for the sulphur and 7500 for the lead. Based on the adhesion of the cementing material to the stone, the unit resistance at the yield-point is 408 lb. for the cement, 328 for the sulphur, and 200 for the lead. In one case the bolt broke, the setting being in cement, and the ultimate resistance 30,000 lb., although the first sign of yielding occurred at a load of 19,500 lb. The superiority of a cement setting is clearly demonstrated by these tests.

Lead filling tends to promote electrolytic action, which may result in a rapid corrosion of the metal. Experience has shown that sulphur filling may also cause rusting of the metal, as well as disintegration of stone or concrete.

The results of other experiments on the adhesion of rods or bars in cement or concrete are given in various treatises on concrete construction.

Various patented forms of anchor bolts, some of which are of the expansion type, are advertised in architectural and engineering periodicals. One of these has the novel feature of being hollow, and was first used on the regulating works of the Chicago Drainage Canal. Holes are drilled in the masonry the same as for ordinary anchor bolts, and then the hollow bolts are attached to the ironwork, which is lined up to final position. The edges of the ironwork joining the masonry surface being pointed with stiff mortar, a thick grout of neat portland cement is forced through the bolt by a hand pump. In this way the ironwork can be set in the final position before the anchor bolts are cemented, and no play is required in the bolt holes of the ironwork. The latter feature is especially valuable where shearing forces are to be opposed. The safe holding power of these bolts may be computed for tension at 15,000 lb. per square inch of net section for the iron, and at 200 to 250 lb. per square inch of embedded bolt surface.

Another type consists of a split bushing with a thread on the inside and projecting points on the outside to be used in securing a screw bolt to stone or brick masonry. An illustrated description may be found in Engineering News, vol. 39, page 286, May 5, 1898. A modification of the same type, having serrated sleeves with a lag screw inserted between, is especially intended to anchor into timber. See Engineering News, vol. 49, page 457, May 21, 1903.

For other designs of expansion bolts the reader is referred to Engineering News, vol. 69, page 316, Feb. 13, 1913, and to Engineering Record, vol. 72, page 122, July 24, 1915.

Problem 18. To what depth is a $\frac{3}{4}$-in. anchor bolt to be embedded in a concrete wall, in order to develop its working stress of 15,000 lb. per sq. in.? The allowable value of the adhesion to be taken is one-fourth of the ultimate value.

Art. 19. Metal Straps and Plates

In many kinds of joints some fastening is necessary to hold the timbers together. For this purpose bolts are most frequently employed, though sometimes spikes, lag screws, or other metal fastenings are used. Metal straps are used in some cases where the simpler fastenings are not conveniently applied, or where the tendency to displacement may require a stronger connection. Figure 19*b* shows a strap holding the housed end of a horizontal brace to the post, the strap being bolted to the brace. In Fig. 19*c* the strap holds the lower chord of a roof truss from sagging. An alternative arrangement is shown in Fig. 19*d*, where a strap of a T-form is attached to each side with lag screws. Sometimes more elaborate and artistic forms are used on exposed roof trusses. Straps are also employed on lock gates and on heavy doors subjected to shocks.

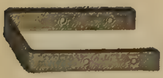


Fig. 19*a*.

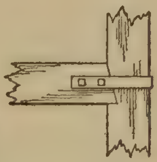


Fig. 19*b*.

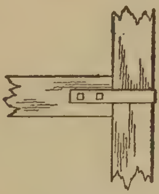


Fig. 19*c*.

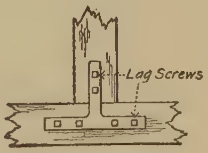


Fig. 19*d*.

Metal straps are used to a limited extent in the framing of wooden bridge trestles. The report of a Committee of the Association of Railway Superintendents of Bridges and Buildings includes the following paragraph: "The fifth plan is that of using bent metal straps, usually called pile straps, being straps about $\frac{1}{2}$ by $2\frac{1}{2}$ in. by 5 ft. in length, bent to engage the trestle caps or sills with legs 2 ft. in length on either side of the post, and secured with heavy spikes to the cap or sill and posts. The fastening, for pile trestles more especially, is being used by some of the best railroads with good results, and, being easily removed when the timbers become defective, it is considered a good device. In using it to secure the caps of pile trestles, the piles, after being cut off to receive the caps, should be squared or sized down to fit snugly the dapping or boxing in the lower side of the caps, which should be made at least one inch in depth. No dowels should be used except in case of a refractory pile which could not be held in place without them. This plan of securing the caps for pile trestle bents makes a secure joint, and admits of easy and speedy renewal of timbers when renewals become necessary."

In Fig. 19*e* straps are used in connection with stepping and housing on a track buffer. As indicated, the ends of two straps are turned inward to form lugs to engage slots cut in the timber, but it is questionable whether they add materially to the strength of the connection, since they are so close to the bolts. In Fig. 19*f* the bent strap has an eye forged on each end to engage a pin or bolt of relatively large diameter. This form is called a heel strap and is occasionally used on the end joints of roof trusses. Whether a heel strap may be expected to resist any part of the horizontal thrust of the upper chord will be discussed in Art. 37.

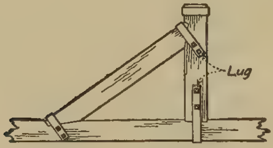


FIG. 19*e*.—Track Buffer.

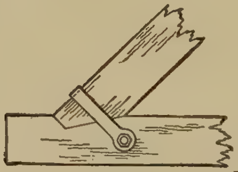


FIG. 19*f*.—Heel Strap.

In Figs. 19*g* and *h* the straps or plates are bent around an angle formed by the timbers to which they are bolted. The former joint is at the corner of a car underframe; and the latter is one of the joints of a foundry crane which has been in service for many years, but its joints appear to be as firm as when built.

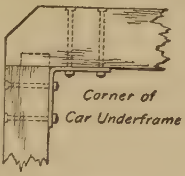


FIG. 19*g*.

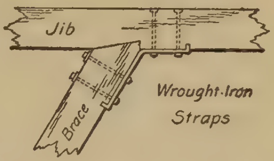


FIG. 19*h*.—Joint in Jib Crane.

Figure 19*i* indicates how metal plates are sometimes used in connecting the timbers of a building frame. A number of straps and plates are shown on the plans of a derrick and tower in Engineering News, vol. 43, page 164 (inset), March 8, 1900, and in Engineering Record, vol. 41, page 576, June 16, 1900.

Since 1907 steel straps and plates have been adopted in the connections of heavy falsework for bridge erection, having wooden posts and horizontal struts, combined with adjustable diagonal rods. In building up the towers all the stories are made of standard lengths, except the one at the top, and are so connected with bolts and pins that they can be taken down and shifted to other places.

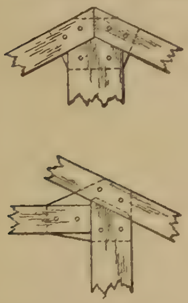


FIG. 19*i*.—Metal Plates.

The application of bent metal plates to wooden trestles is illustrated in Fig. 19*j*, the two left-hand diagrams showing the details more clearly in isometric projection. Their construction

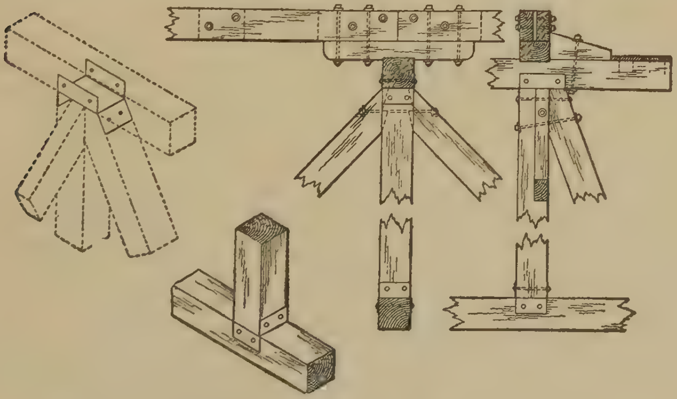
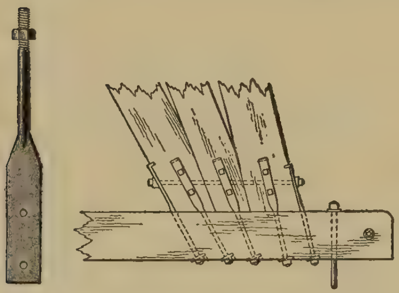


FIG. 19*j*.—Wrought-iron Plates in Trestle Construction.

and advantages are described as follows in the report of the committee previously referred to in this article: "The fourth method, that of using bent metal plates, has given very good results. These plates are made of thin boiler-plate iron, generally one-fourth of an inch in thickness, cut and bent with flanges in opposite directions at right angles to opposite sides of the plates, one set of flanges engaging the cap or sill, and the other set engaging the posts of the trestle bent. They can be rapidly and cheaply made to suit any sized timber and any angle at which the timbers meet, leaving but little work to do in the field; namely, that of squaring the timber and sizing it down to fit between the flanges on the plates, and spiking them in place. This method produces a joint of considerable strength, and is free from many of the objectionable features found in the dowel and drift-bolt joints."



Figs. 19*k* and *l*.—Strap Bolts.

Strap bolts consist of a strap at one end and a bolt at the other as shown in Fig. 19*k*. The detail in Fig. 19*l* shows their application in fastening the back braces of a piledriver to the bed timbers. The ends of the straps are bent inward to form a lug.

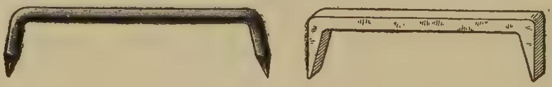


Fig. 19*m*.—Cramps.

A cramp or iron dog, illustrated in Fig. 19*m*, is a piece of round or square iron about $\frac{3}{4}$ in. in diameter, with ends pointed or chiseled and turned down about 3 in., serving to hold two pieces together. It may be used either to bind together the ends of the two pieces, as in a butt joint, or the end of one piece to the side of another. A cramp is also used in masonry to bind adjacent stones together where great strength is required, as in the masonry of the pivot pier of a swing bridge or in a lighthouse.

Problem 19. Explain the method of designing a strap bolt and its connections in the end of a timber, to resist a given tensile stress.

CHAPTER II

JOINTS USED IN FRAMING

ART. 20. TABLED FISH-PLATE JOINT

TIMBERS are joined in the direction of their length by the operation called splicing. The joints so formed are divided into three classes: lapping, fishing, and scarfing. When a timber is to transmit only tension, fishing is more effective than either lapping or scarfing.

The best form of fishing for this purpose is that containing tabled fish plates, illustrated in Fig. 20*a*. The plates are also called indented fish plates. The projecting tables of the fish plates engage corresponding notches in the main timbers. The bolts are required to keep the parts in position and to resist the tendency of the fish plates to bend.

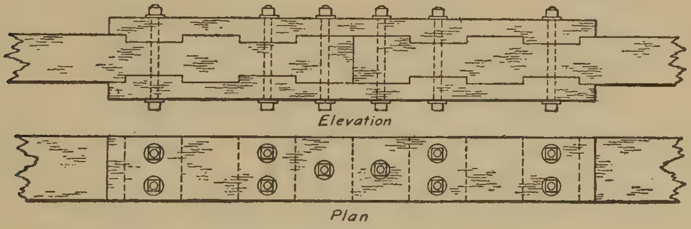


FIG. 20*a*.—Tabled Fish-plate Joint.

If such a joint is tested to destruction it may fail: first, by the rupture of one or both of the main timbers; second, by the rupture of one or both of the fish plates; third, by the rupture of one or more of the bolts, which in turn would cause the failure of the fish plates by flexure; or fourth, by one or more washers crushing the fibers of the fish plates and thus causing their failure by splitting or tearing. A main timber may fail: first, by tearing the fibers in the smallest cross-section which transmits the full stress, and this occurs at the bolts near the ends of the fish plates;

second, by crushing the ends of the fibers on one side of each table; or third, by shearing the tables at their bases, along the fibers. A fish plate may fail in the same manner. The failure of bolts of standard dimensions (see Art. 1) occurs by rupture at the root of the thread.

The joint will be safe when all its parts are so designed that there is safety in every one of the respects named. Because the strength of a joint is equal to that of its weakest part, good design requires an equal degree of security in all of these respects. Since one part affects another, it is necessary to consider their mutual relations, together with the conditions involved in the actual construction of the joint.

THE NET TENSILE SECTION

As the safe strength of the net section of the main timber must equal the given stress to be transmitted by the joint, economy requires that the gross section shall be no larger than that needed to furnish also the tables, the combined strength of which equals the given stress. If the notches are too deep, there is a waste of timber the relative value of which depends upon the length of the timbers spliced. In practice the cost of two or more designs may be compared and the most economical one adopted, the cost of both material and workmanship being taken into account.

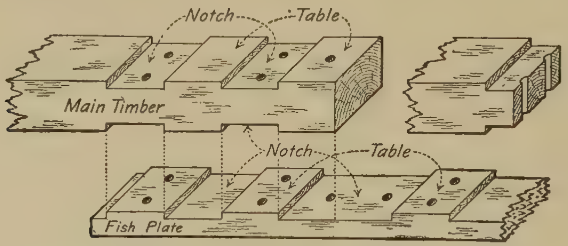


Fig. 20*b*.—Parts Separated to Show Notching.

It will be noticed from Fig. 20*b* that the plane containing the net section area of the main timber passes through two bolt holes, whereas that of each fish plate passes through only one. The two bolts next to the center of the joint are inserted to keep the parts in contact and to strengthen the joint while being handled and placed in its final position in the structure.

THE NUMBER OF TABLES

The number of tables on each main timber as well as the number of those on each fish plate must evidently be an even number, and hence it is necessary to decide whether 2, 4, or 6 are to be adopted. The cost of workmanship increases almost directly as the number of tables. On the other hand, the amount of timber decreases somewhat as the number of tables increases, since the height of the tables is thereby reduced. The ratio of the amount of material to the number of tables depends also on the length of the timbers to be spliced, and is therefore not a simple one. On account of imperfections in workmanship, the larger the number of tables the less likely they are to work together, or to receive equal stresses. If any table receives a larger stress than its share, the degree of security of the entire joint is thereby reduced.

Further, the slight inequality in the depths of the tables on the main timbers and of the corresponding ones on the fish plates, due also to defective workmanship, diminishes their effective bearing area, and this reduction is relatively greater in low tables than in high ones. Only in fine work can a carpenter be expected to work as close as a sixteenth of an inch, whereas in large framed joints the depths are usually expressed only to a full eighth of an inch. On the other hand, if the number of tables is reduced, the resultant line of stress is carried farther from the axis of the main timbers. This increased deviation implies the development of larger bending moments to be resisted by the bolts, and their increased size reduces in turn the net tensile section of the timber.

Again, the length of the tables is reduced by increasing their number. If the bearing is not uniform across the end of each table, the strength of the entire joint is thereby reduced; and if the joint were tested to destruction, it might fail by shearing off the tables by parts in succession without developing their full strength under proper conditions. To avoid this reduction it is advisable to limit the ratio of the length of a table to its width, and this ratio ought probably to be not less than two-thirds. On the other hand, the length should not be greater than about the width of the timber, in order that the bolts may readily keep the parts in contact.

THE BOLTS

The bolts do not simply keep the parts in position, but they receive tensile stresses that depend directly upon the magnitude of the stress carried by the joint. Since the resultant of the tension in the net section of the fish plate does not coincide with that of the compression against the adjacent table, a moment is developed which tends to bend the fish plates. This moment is resisted by the bolts at each table, the center of rotation being in the surface of contact between the table of the fish plate and that of the main timber. It will be observed that in this surface the ends of the fibers engage one another.

The farther the bolt is from the center of rotation the less will be its tensile stress, and hence the smaller its diameter. In order that the joint shall not open between the fish plates and main timber when the joint is under its maximum stress the bolts must be drawn up to their full tension when the joint is constructed. If the bolt is not at the mid-length of the table, the fibers on its surface will be compressed unequally and thus introduce initial bending in the fish plates. It is, therefore, preferable to place the bolts at, or very near to, the mid-length of the table, even though it requires slightly larger bolts than if placed near the farther end. If by placing a bolt just a little beyond the middle, its full safe strength can be utilized, it is sometimes preferable to using a larger bolt exactly at the middle which may have considerable excess of strength, although the latter may at the same time decrease the net section just enough to require an addition of a full inch to the gross depth of the timber.

Since the stress in a bolt is transmitted to a fish plate by the bearing of a washer, it is evident that there must be a limit to the width of fish plate which can be held effectively without warping or without transmitting the stresses too indirectly. If the timber is not wider than 6 in., one row of bolts will answer. Two rows are used for a greater width, and three rows when the width exceeds 14 in. When two rows of bolts are used, it should be assumed that each row acts on its half of the width of the timber, and should be located on the center line of that half; and similarly for three rows.

To insure this condition the holes should be bored from $\frac{1}{16}$ to $\frac{1}{8}$ in. larger than the diameter of the bolts. The bearing area of the washer is to be computed. It should also be observed that the diameter of the hole in the washer is larger than that of the bolt (see Art. 2). The thickness of the washer may then be selected from the tables of standard or special sizes manufactured for the trade.

CONTROLLING CONDITIONS

Whenever the structure imposes any controlling conditions on the proportions of the joint, some of the preceding considerations may have to be waived. For instance, if the joint must be shorter than the length previously determined, it can be made so only by increasing the width, since the shearing area is fixed. The slight extensions of the fish plates beyond their end tables are made to prevent the outer fibers of the main timbers from splitting off readily.

APPLICATIONS

Figure 20*c* shows a splice in the sill of a trestle bent, in which the fish plates have only two tables. The sill is subject to only a small tensile stress, due to the inclination of the batter posts, its main function being to act as a beam to distribute the load from the posts to the foundations.

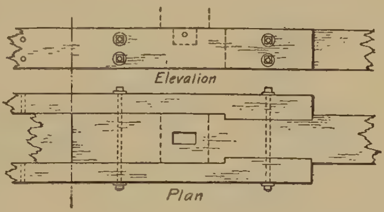


FIG. 20*c*.—Splice in Sill of Trestle Bent.

In Fig. 20*d* is indicated a splice in the lower chord of a roof truss of an exposition building. The two white pine timbers break joints so that only one needs to be spliced at any given section. The oak fish plate also acts as a packing block. Since there are tables on only one side of the timber, flexure is developed which causes additional stresses in the fish plate and other chord timber. The shorter tables on the fish plate indicate the greater resistance of oak to shearing along the grain.

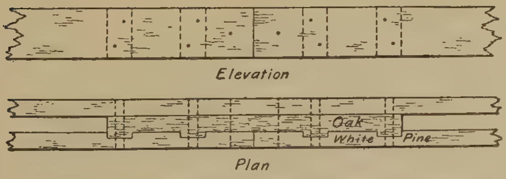


FIG. 20*d*.—Splice in Lower Chord of Truss.

Sometimes the tabled fish-plate joint is modified by cutting notches opposite to each other in both the fish plates and main timber and inserting transverse keys. An example of this kind may be seen in Fig. 72*e*. It is less efficient than the regular form on account of the tendency of the keys to rotate and the smaller resistance to compression on the side of the fibers. It is similar to the shear-pin splice (Art. 29) in action, but lacks the simplicity of construction of the latter.

The indented fish-plate joint in Fig. 20*e* is shorter than the tabled joint with the same number of tables, but requires more careful workmanship and very straight-grained timber. Although the shearing surface of the table extends, theoretically, over its full length, it cannot be so regarded practically on account of the tendency to inclined shear near the thin edge of the table. Another advantage lies in the fact that the bolts are neither in the shallowest section of the main timber nor of the fish plates.

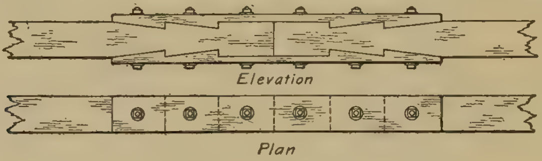


Fig. 20*e*.—Indented Fish-plate Joint.

Problem 20. Compute the approximate relation between the lengths of the tables in Fig. 20*d*, provided the working unit stresses adopted by the American Railway Engineering Association are specified (Art. 89).

Art. 21. Design of Tabled Fish-plate Joint

The following order of design is suggested as a guide to the student who is just beginning to apply the principles of mechanics to the design of structures. Its aid simplifies the computations, avoids unnecessary revisions, and secures a systematic arrangement of the work.

Order of Design: 1. Compute the required areas for tension, compression, and shear. 2. Assume the number of tables on each of the main timbers. Usually four tables give the best results.

3. Determine the width of the timber and the approximate lengths of the tables. 4. Assume the diameter of the bolts after observing those on similar designs, and find the net and gross depth of the main timbers. 5. In case different material is used for the fish plates some of the computations indicated above are to be repeated for the fish plates. 6. Find the required net section area of the bolts, as well as the required bearing area and diameter of the washers. Consult the tables of standard bolts and washers. If no standard washer meets the requirements, design one for the given case. Record the net areas furnished by the bolts and washers adopted. 7. Compare the computed and assumed diameters of the bolts and revise the gross section of the timber if necessary. 8. If the gross depth of the timber adopted exceeds that required, decide whether to include the excess either in the net depth or in the height of tables, or to divide the excess so as to increase both the tensile and compressive areas. State the reason governing the decision. 9. Make an alternative design by taking a different width to learn whether a more economical section of the timber may be secured, or the bolts may be reduced in size or length. 10. Prepare a bill of material. 11. Make an estimate of cost. 12. Make a working drawing of the joint, including a side elevation and a plan drawn to a scale of $1\frac{1}{2}$ in. to the foot.

For example, let it be required to design a tabled fish-plate joint to transmit a tension of 64,000 lb. The timber to be used is dense select southern pine and the joint is in a building protected from the weather, as in the lower chord of a roof truss. Let the unit working stresses be specified as follows in accordance with Arts. 89 and 90: For tension, 1750 lb.; compression with the grain, end bearing, 1713 lb.; compression across the grain, 380 lb.; and shear along the grain, 256 lb. Let the unit tensile stress in the bolts be taken at 16,000 lb. Remembering the principles given in Art. 20 and the suggested order of design, the following computations are made: The net tensile area is $64{,}000/1750 = 36.5$ sq. in.; the compressive area is $64{,}000/1713 = 37.3$ sq. in.; and the shearing area is $64{,}000/256 = 250$ sq. in. If four tables are adopted for each timber, the shearing area of one table is $250/4 = 62.5$ sq. in. In order that the length of the table shall not be less than two-thirds of its width and may not materially exceed its width, the dressed width of the timber will be taken at $9\frac{1}{2}$ in.

The length of the table must slightly exceed 62.5/9.5 = 6.58, say, $6\frac{3}{4}$ in., in order to allow for two bolt holes.

Let the diameter of the bolts be assumed temporarily as $\frac{3}{4}$ in., while the diameter of the bolt hole is made $\frac{1}{8}$ in. larger. The net width of the tensile section is therefore $9.5 - 2 \times \frac{7}{8} = 7\frac{3}{4}$ in. and the net depth of the main timber is 36.5/7.75 = 4.71, say, 5 in. The depth of the tables must be $37.3/(4 \times 9.5) = 0.98$ in. in order to furnish the necessary bearing area. Using a depth of $1\frac{1}{4}$ in. makes the gross depth of the timber $5 + 2 \times 1.25 = 7.50$ in. dressed thickness. Hence, the nominal section will be 8 by 10 in. (Art. 83).

Since the same kind of wood is used for the fish plates as for the main timbers, the stress in each plate is one-half of that transmitted by the joint, and hence both its net and gross depths are one-half of those of the main timbers. Considering only the stress carried by one table of the fish plate, the lever arm of the couple of the resultants of the tension in the net section of the fish plate and of the corresponding compression in the end of the table is one-half of the gross depth of the fish plate, and therefore the moment of rotation of this couple is $(\frac{1}{4} \times 64{,}000)(\frac{1}{2} \times 3.75) =$ 30,000 in.-lb. The width of the timber requires two bolts at each table (Art. 20). The lever arm of the resultant pressure under the washer for each bolt, if placed at the center of the table, about the center of rotation in the end of the table is at least one-half of the length of the table, or $6.75 \div 2 = 3.37$ in. Hence, the stress in one bolt is $\dfrac{\frac{1}{2} \times 30{,}000}{3.37} = 4450$ lb., and the required area of the bolt at the root of the thread is 4450/16,000 = 0.278 sq. in. On reference to the manufacturer's handbook (or Art. 1) it is found that the diameter of the bolt needed is $\frac{3}{4}$ in. and its net section area is 0.302 sq. in. As this agrees with the size assumed, no revision of the net depth is necessary. The section area of a $\frac{7}{8}$-in. hole is 0.601 sq. in., and by adding two such areas to the required shearing area of the table, its length is found to be $63.7 \div 9.5 = 6.70$ in.

The required bearing area of the washer is $4450 \div 380 =$ 11.72 sq. in., and the area of its $\frac{7}{8}$-in. hole is 0.60 sq. in. By reference to a handbook, the diameter of a circle having an area of not less than 12.32 sq. in. is found to be 4 in.

The detail drawing for this design, with all the dimensions required for its construction, is given in Fig. 21*a*. The bill of

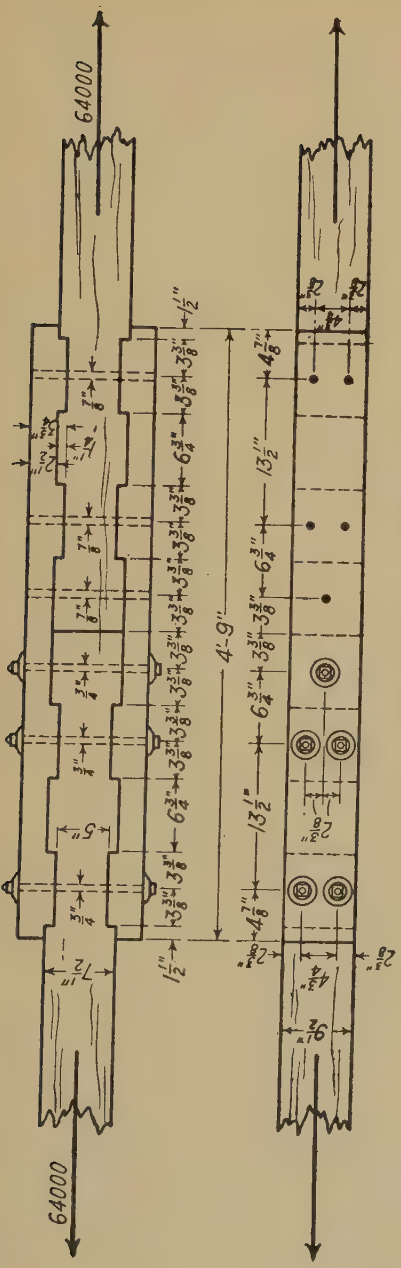

64000
64000
4'-9"
$13\frac{1}{2}"$
$6\frac{3}{4}"$
$4\frac{7}{8}"$
$7\frac{1}{2}"$
$9\frac{1}{2}"$

FIG. 21*a*.—Working Drawing of Tabled Fish-plate Joint.

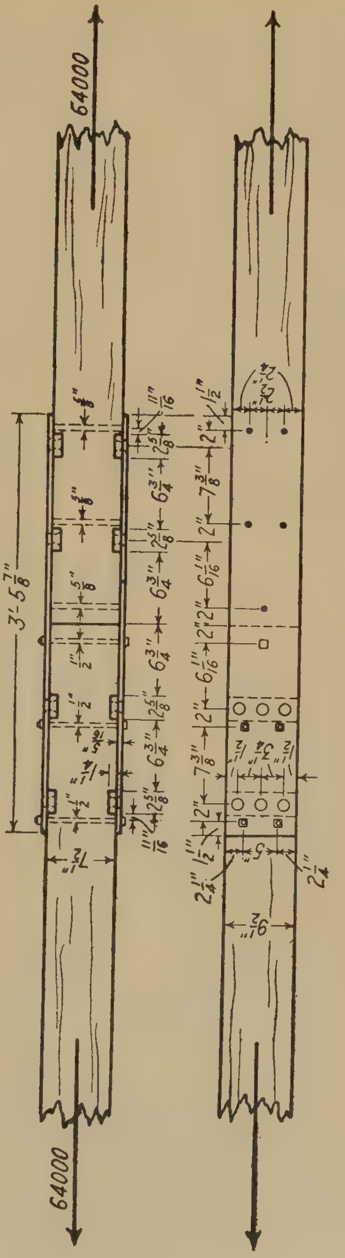

64000
64000
$3'\text{-}5\frac{7}{8}"$
$7\frac{1}{2}"$
$9\frac{1}{2}"$
$2\frac{1}{4}"$

FIG. 21*b*.—Working Drawing of Joint with Steel Tabled Fish Plates.

material for the joint and main timbers, which are assumed to be 20 ft. long, is as follows:

2 main timbers, 8 × 10 in. × 20 ft. 0 in.	266.6 ft. b.m.
2 fish plates, 4 × 10 in. × 4 ft. 9 in. [5 ft. 0 in.]	33.3 ft. b.m.
10 $\frac{3}{4}$-in. bolts, 15$\frac{1}{2}$ in. long; 10 nuts	23.1 lb.
20 ogee washers, 4-in. diam., @ 1.1 lb.	22.0 lb.

The estimate of cost is as follows:

300 ft. b.m. lumber @ 6 cents	$18.00
23.1 bolts and nuts @ 5 cents	1.16
22 lb. washers @ 5 cents	1.10
8 hours' labor @ 80 cents	6.40
	$26.66

Problem 21. Design a tabled fish-plate joint of dense select southern pine to transmit a tensile stress of 32,000 pounds.

Art. 22. Tabled Fish Plates of Steel

Figure 22*a* shows a splice in one of the three timbers composing the lower chord of a roof truss, in which tabled fish plates of steel are employed. The tables on the fish plates consist of "flats," which are riveted to the plates. These tables are often called "ribs." The heads of the rivets are countersunk in the flats to avoid further reduction of the net tensile section of the timber.

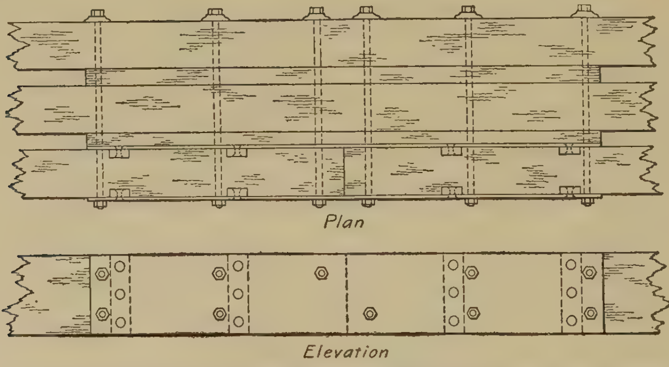


Fig. 22*a*.—Splice in Lower Chord with Steel Tabled Fish Plates.

In designing the fish plates it is necessary to observe the standard specifications for steel structures, which are based on experience. The spacing of rivets, center to center, in the direction of the row, is known as the pitch. The minimum allowable pitch is 3 diameters of the rivet, but it is generally preferable to use a slightly larger value. The minimum distance from the center of a rivet to the rolled edge of a plate, or of a flat, is $1\frac{1}{2}$ and preferably $1\frac{3}{4}$ diameters. The best sizes of rivets for such a joint are $\frac{5}{8}$, $\frac{3}{4}$, and $\frac{7}{8}$ in. in diameter. The standard sizes and proportions of rivets are given in the handbooks of various bridge manufacturers. The minimum thickness of plates should be $\frac{1}{4}$ in. when not exposed to the weather, and $\frac{5}{16}$ in. when so exposed. A limit is fixed on account of the deterioration of the metal, due to rust or corrosive gases. Where corrosion, due to gas, is unusual, an extra addition is made to the thickness of the metal.

In staggered riveting the distance between the rows should not be less than one-half of the pitch of the rivets in a row, plus one-quarter of the diameter of a rivet hole.

Another rule prescribes that this distance shall not be less than 2 diameters of the rivet for staggered riveting, whereas $2\frac{1}{2}$ diameters are to be used for chain riveting.

The distance from the rivet hole to the end of a tension plate should not be less than 80 percent of the net distance between the rivet holes in a row, or the distance from the center line of rivets to the end of the plate should not be less than $1\frac{3}{4}$ diameters of the rivet.

In chain riveting, the rivets in adjacent rows are directly opposite each other, whereas in staggered riveting the rivets in one row are located in intermediate positions with respect to those in the other row.

In designing the fish plates the ribs may be regarded as beams under a uniform load, due to the compression from the ends of the wooden fibers, and with supports at the rivets. Usually, however, the minimum allowable width gives sufficient strength in this regard. The rivets are subject to single shear, and also to bearing both in the rib and the plate. The bearing area of a rivet is the projection of its cylindrical bearing surface upon a diametral plane, and equals the product of the rivet diameter by the thickness of the plate. The thinnest plate through which any rivet passes determines its bearing value. The balance of the design is practically the same as that explained in Arts. 20 and 21. It may be added, however, since the bolts are so close to the steel ribs, that the net section of the timber should be taken the same as if the net depth and the net width occurred in the same cross-section.

PROBLEM 22. Make a list of the tabled fish-plate joints, with metal fish plates, shown in the illustrations in Chapters IV and V.

ART. 23. DESIGN OF TABLED FISH PLATES OF STEEL

Let it be required to design a tabled fish-plate joint with steel fish plates for a tensile stress of 64,000 lb. in order to compare it with the one designed in Art. 21, which has wooden fish plates. The tensile, compressive and shearing areas for the wood are, respectively, 36.5, 37.3, and 250 sq. in., being the same as found in Art. 21. Let the dressed width be taken as before, or $9\frac{1}{2}$ in.

The unit stress to be used for tension in the steel plate and bolts is 16,000 lb. per sq. in., while those for shear and bearing of the rivets are 12,000 and 24,000 lb. per sq. in., respectively. With the aid of a rivet table in a handbook the values for single shear for $\frac{3}{4}$- and $\frac{7}{8}$-in. rivets are found to be 5300 and 7220 lb. As four tables will again be adopted, the pressure on one rib or table of the fish plate is 16,000 lb., which requires either four $\frac{3}{4}$-in. rivets or three $\frac{7}{8}$-in. rivets. We will use three $\frac{7}{8}$-in. rivets, spacing them with a pitch of $3\frac{1}{4}$ in., the outside ones being placed $1\frac{1}{2}$ in. from the edge of the plate.

The net tensile section of the plate required is $\frac{64,000}{2 \times 16,000} = 2.00$ sq. in. The net width of the plate is $9.5 - 3 \times 1 = 6.5$ in., since specifications generally require a deduction for holes $\frac{1}{8}$ in. larger than the diameter of the rivets, in all tension members. The thickness of the fish plate is $2.00 \div 6.5 = 0.308$ in., or $\frac{5}{16}$ in. As the bearing of a $\frac{7}{8}$-in. rivet in a $\frac{5}{16}$-in. plate is 6560 lb. (see handbook) for the specified unit stress, the rivets are also safe in bearing, and no revision is necesary.

According to the practical requirements given in Art. 22, the rib must be at least $2 \times 1\frac{1}{2} \times \frac{7}{8} = 2\frac{5}{8}$ in. wide. The required thickness of the rib is 1 in., or the same as the required height of table for the wooden fish plate. Since the flat is thicker than the diameter of the rivets, the holes should be drilled.

Let the centers of bolts be placed 2 in. from the center line of the rivets or of the rib, making the lever arm for the tension in the bolts $2 + \frac{1}{2}(2.625) = 3.312$ in. The moment of rotation for the rib is $16{,}000 \times \frac{1}{2}(1.0 + 0.313) = 10{,}500$ in.-lb. The tension in each of the two bolts is $\frac{10{,}500}{2 \times 3.313} = 1586$ lb., and the area required at the root of the thread is $1586 \div 16{,}000 = .0992$ sq. in. This requires a $\frac{1}{2}$-in. bolt, which has a net area of 0.126 sq. in. The bolt holes in the plate should be $\frac{1}{16}$ in. larger than the bolts, and the distance from their centers to the end of the fish plate is made $1\frac{1}{2}$ in. (Art. 22).

The net width of the timber is $9.5 - (2 \times \frac{5}{8}) = 8.25$ in., and the net depth required is therefore $36.5/8.25 = 4.43$, or $4\frac{1}{2}$ in., making the required gross depth $4\frac{1}{2} + 2 \times 1 = 6\frac{1}{2}$ in. The depth will be made $7\frac{1}{2}$ in. in order to use a commercial size. The extra inch will be divided between the tensile and bearing areas, the ribs being made $1\frac{1}{4}$ in. thick and the net depth 5 in. The length of the table is $\frac{(62.5 + 2 \times 0.307)}{9.5} = 6.64$, or $6\frac{3}{4}$ in., since the area of a $\frac{5}{8}$-in. hole is 0.307 sq. in. The total length of a fish plate is

$$2(2 \times 6\tfrac{3}{4} + 1\tfrac{1}{2} \times 2\tfrac{5}{8} + 2 + 1\tfrac{1}{2}) = 41\tfrac{7}{8} \text{ in.}$$

The detail drawing for this design, with all the necessary dimensions, is given in Fig. 21*b*. The bill of material for the joint and main timbers, assumed to be 20 ft. long, is as follows:

2 main timbers, 8 × 10 in. × 20 ft. 0 in.	266.6 ft. b.m.
2 steel plates, $9\frac{1}{2} \times \frac{5}{16}$ in. × 3 ft. $5\frac{7}{8}$ in.	70.5 lb.
8 flats, $2\frac{5}{8} \times 1\frac{1}{4}$ in. × $9\frac{1}{2}$ in.	70.4 lb.
10 $\frac{1}{2}$-in. bolts, $8\frac{3}{4}$ in. long; 10 nuts	6.2 lb.

The estimate of cost is as follows:

267 ft. b.m. lumber @ 6 cents	$16.02
141 lb. steel plates (riveted) @ 6 cents	8.45
7 lb. bolts and nuts @ 6 cents	0.42
5 hr. labor @ 80 cents	4.00
	$28.89

PROBLEM 23. Compute the working strength of the joint in the lower chord of the upper roof truss shown in Plate V.

ART. 24. PRESSURE OF WOOD ON METAL PINS

In framing, it is sometimes necessary to use metal pins or bolts as beams in transferring stresses from one timber to another. This involves the determination of the pressure of the fibers of the wood upon the cylindrical surface of the pin. When the resultant of the pressure is perpendicular to the fibers of the wood, the magnitude of the resultant is the same as if the bearing surface were the diametrical section of the pin. But when the direction of the resultant is parallel to the fibers of the wood, the case is entirely different because the resistance of the fibers to lateral compression is much less than to longitudinal compression.

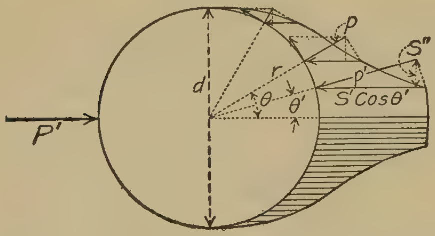


FIG. 24*a*.

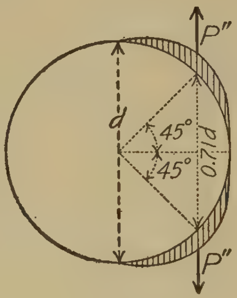


FIG. 24*b*.

Theoretically, the resultant pressure may be determined approximately in the following manner: Let s be the longitudinal unit pressure which is parallel to the fiber, p the unit pressure normal to the surface of the pin, θ the angle which it makes with the direction of the fibers, as indicated in Fig. 24*a*, r the radius and d the diameter of the pin or bolt, and h the height of the timber bearing against the pin. Let s' be the safe unit-stress for compression parallel to the fiber, that is, for bearing on the ends of the fibers, and s'' for bearing on the side of the fibers.

The normal unit pressure p is the component of the longitudinal unit pressure s, or $p = s \cos \theta$. It may be assumed that s equals s' for values of θ from 0 to θ', the latter angle being that for which the transverse component of p equals s''. Accordingly, $s'' = s' \cos \theta' \sin \theta'$ or $\sin^2 \theta' = 2\ s''/s'$. The transverse component of p must then gradually diminish until it becomes zero when $\theta = 90$ deg. Let it be assumed to vary directly as $\cos \theta$. Its value is then $s'' \cos\theta/\cos\theta'$, from which is found $p = s'' \cos\theta/\cos\theta' \sin\theta$ and $s = s''/\cos\theta' \sin\theta = s' \sin\theta'/\sin\theta$, between the limits of θ' and 90 deg.

Let the coordinate y of any point on the circumference be perpendicular to the fibers of the wood, then $y = r \sin\theta$ and $dy = r \cos\theta \, d\theta$. The pressure on the area hdy is $shdy$, and by integrating this expression between the limits of + 90 deg., and − 90 deg., the resultant longitudinal pressure on the pin is found to be

$$P' = hds' \int_0^{\theta'} \cos\theta \, d\theta + hd\, s' \sin\theta' \int_{\theta'}^{90} \cot\theta \, d\theta$$

$$= hds'[\sin\theta'(1 - \log_e \sin\theta')] \tag{1}$$

For wood having a ratio of 0.25 between the safe unit bearing on the side and on the end of the fibers respectively, $\theta' = 15$ deg., and

$$P = 0.62\, hds' \tag{2}$$

This result indicates that the safe longitudinal pressure of such timber on the side of a round pin is approximately six-tenths of the corresponding pressure on a square pin of the same diameter. When the ratio referred to is reduced to 0.20, the numerical coefficient in (2) becomes 0.53.

An experimental determination for longleaf yellow pine, but one in which the timber was tested to its ultimate strength, gave an average coefficient of 0.63 for 5 tests. The specimens were prevented from splitting by means of clamps. The plane of division between the fibers crushed sidewise was marked in every case and gave an average value for θ' of $15\frac{1}{2}$ deg.

The transverse components of the normal pressures on each half of the pin are respectively equal and opposite to those on the other side, and together they tend to split the timber (Fig. 24*b*). The transverse component of P is $s' \cos\theta \sin\theta$ for values of θ between 0 and θ', and is $s'' \cos\theta/\cos\theta' = s' \cos\theta' \sin\theta' \cos\theta$ between θ' and 90 deg. Let the coordinate x of any point on the circumference be parallel to the fibers, then $x = r \cos\theta$ and $dx = - r \sin\theta \, d\theta$. The negative sign may be neglected since it is immaterial whether the horizontal side of the rectangle hdx is measured in a positive or negative direction to obtain its area. The resultant transverse pressure on the pin is

$$P'' = \tfrac{1}{2}\, hds' \int_0^{\theta'} \cos\theta \sin^2\theta \, d\theta + \tfrac{1}{2}\, hds' \cos\theta' \sin\theta' \int_{\theta'}^{90°} \cos\theta \, \sin\theta \, d\theta$$

$$= \tfrac{1}{2}\, hds'(\tfrac{1}{3}\sin^3\theta' + \tfrac{1}{4}\sin 2\,\theta' \cos^2\theta'). \qquad (3)$$

For wood in which the ratio of s'' to s' is 0.25, equation (3) becomes $P'' = 0.061\ hds'$; and substituting the value of s' from (2) there is obtained the relation

$$P'' = 0.10\, P'. \qquad (4)$$

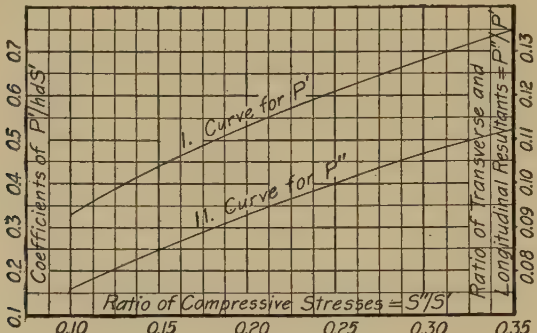


Fig. 24c.

In other words, the transverse stress which must be resisted by tension across the fiber is approximately one-tenth of the longitudinal pressure on the pin. For a ratio of $s''/s' = 0.20$ the coefficient in (4) reduces to 0.093. It may be of interest to add that the resultant P'' is applied at a point for which θ is about 45 deg. Figure 24*c* gives two curves from which the numerical coefficients in equations (2) and (4) may be found directly, when the allowable unit-stresses s'' and s' have been determined for the given species of wood to be used in the construction.

When the resultant of the pressure of the wood on a round pin is perpendicular to the fibers, the magnitude of the safe bearing value is to be taken as hds''; that is, the pressure is the same as if the pin were square or rectangular in cross-section.

Problem 24. If the allowable unit compressive stresses on the side and ends of the fibers of white oak are respectively 450 and 1300 pounds per square inch, it is required to compute the values of P' and P''.

ART. 25. PLAIN FISH-PLATE JOINT

In this joint the main timbers are spliced end to end by means of two plain fish plates and connecting bolts, as shown in Fig. 25*a*. The stress is transmitted from each main timber to the fish plates through the bolts acting as beams.

The stress is first carried by the continuous fibers past the bolts, and then transferred to the fibers directly behind the bolts, through their lateral cohesion or the shearing strength parallel to the fibers. The pressure of the ends of these fibers constitutes the load on the bolt beam, whereas the reactions consist of the corresponding pressures of the fish plates.

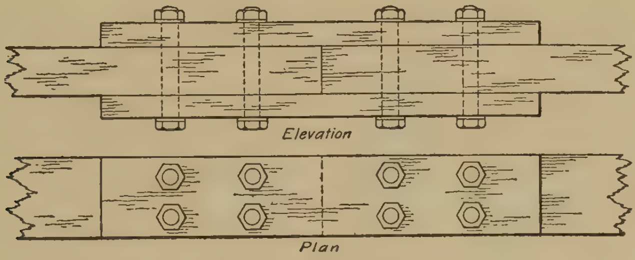


FIG. 25*a*.—Plain Fish-plate Joint.

Theoretically, the best arrangement would be to use square bolts with their sides respectively parallel and perpendicular to the fibers of the wood, since the flexural strength of a square bolt is 67 percent larger than that of a round bolt of the same diameter, whereas the corresponding effective bearing areas of the wood may differ by a percentage ranging from about 25 to over 60. Practical difficulties in construction, which materially increase the cost, have hitherto prevented the adoption of square bolts.

In the design of the plain fish-plate joint the following facts and principles must be kept in mind: First, the reduction in the effective bearing area of the wood against a bolt, on account of the cylindrical surface of the latter, as discussed in the preceding article. In general, an increase in the compressive area requires an increase in the number of bolts, but occasionally a slight increase may be obtained by increasing the diameter of the bolts. Second, the wood tends to shear in two surfaces not tangent to the bolt, but closer together on account of the unequal pressure around the surface of the bolt. In spacing the bolts center to center longitudinally, it is best to add the full diameter to the length of the shearing surfaces required. Third, the transverse components of the pressure on the bolts tends to split the timber at or near the axial plane of the bolts. As this surface is so close to the shearing surface they naturally tend to combine, especially if the timber is not entirely straight-grained. It is on the side of safety, therefore, to add the required length of the splitting surface to that of each shearing surface, since it may combine with either one. Sometimes small transverse bolts are inserted to resist the tendency to split the timber. Their use shortens the length of the joint, but increases its cross-section. Fourth, on account of the low shearing strength of wood parallel to the grain, the bolts are placed directly opposite when more than one row is used with wooden fish plates.

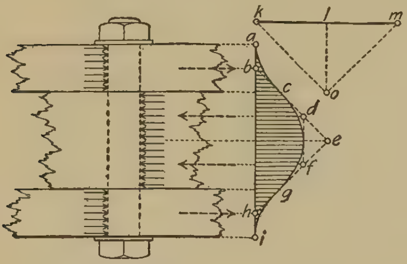


Fig. 25*b*.—Bending Moments in Bolt.

Fifth, according to mechanics, the distribution of the bending moments in a bolt is that illustrated in Fig. 25*b*, when it is assumed that the pressure on the bolt is uniformly distributed along its length, as indicated by the full arrows. The curves *ac*, *cg*, and *gi* are parabolas. When the pressures in each fish plate, and in each half of the main timber, are replaced by their respective resultants, as shown by the broken arrows, the moment diagram becomes the polygon *bdfh*, in which the maximum ordinate is the same as before. Accordingly, the maximum bending moment equals the stress in the fish plate multiplied by the distance from the center of the fish plate to the quarter point of the main timber. Since the bending moment varies directly as the depth of the timber, it is desirable to keep the depth as small as practicable while maintaining a fair relation between the depth and width. If the fish plate is too wide in proportion to its thickness, it is more liable to warp. The resisting moment of a bolt equals $f\pi d^3/32$, in which f is the stress in the outer fiber and d the diameter. The strength of the bolt, therefore, varies as the cube of the diameter, and for a given change in resisting moment the number of bolts changes rapidly in proportion to any change in diameter.

Where the fish plates and main timbers are of considerable thickness, tests indicate that the pressure is not uniformly distributed along the bolts, having a maximum intensity on the portions of the bolts immediately adjacent to the contact faces of the timbers.

For purposes of design the pressure may be assumed uniformly distributed over a distance, a, each way from the contact surface such that the resultant bending in the bolt will just equal the flexural strength of the bolt.

The value of a is given by the formula:

$$a = d\left(\frac{\pi f}{32s}\right)^{1/2}$$

where d = diameter of bolt in inches, f = allowable flexural unit stress in the bolt, and s = allowable unit bearing stress on the wood fibers. The moment in the bolt will be $\frac{Pa}{2}$. This formula may be used wherever it gives a value less than that found by using the method explained in the preceding paragraph.

In the plain fish-plate joint the bolts are not subject to any appreciable tension which requires consideration in designing, since the nuts are only drawn up lightly to bring the fish plates into contact with the main timbers. On this account it is not necessary for the bolts to have the standard size nut and head of common bolts. The joint will also appear clumsy if these standards are used. For this purpose lateral pins used in bridges, but with shallow nuts instead of cotters, are well adapted, the standard sizes of which are given in the Cambria steel handbook and in the handbooks of some other steel manufacturers. When wooden fish plates are used so that the bearing is against the wood only, the nominal diameter may be used, thus avoiding the necessity of turning down the pin to the finished diameter as required for bearing against steel. Another arrangement is to insert a tap bolt and washer at each end of the bolt as indicated in Fig. 25c.

In some cases it may be a more economical arrangement to employ double extra hydraulic tubing for the pins, and use small bolts and washers to hold the parts securely in place. A tube with outside and inside diameters of 1.31 and 0.58 in., respectively, has 96.1 percent of the flexural strength of a solid bolt of the same external diameter. For corresponding diameters of 1.66 and 0.88 in. the percentage is 92.2; for diameters of 1.90 and 1.08 in. the percentage is 89.5; and for diameters of 2.37 and 1.49 in. the percentage is 84.5.

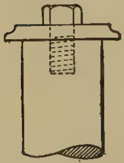


Fig. 25*c*.

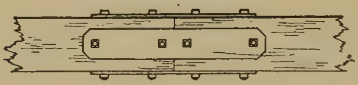


Fig. 25*d*.—Splice in Post.

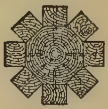


Fig. 25*e*.

For compression members this form is preferred to the tabled fish-plate joint. In this case also the fish plates are put on the four sides of the stick as shown in Fig. 25*d*, and the bolts are much smaller than for a tension splice, the object of the fish plates being merely to prevent deflection in the column and not to take any part of the direct compression. In Fig. 25*f* is shown

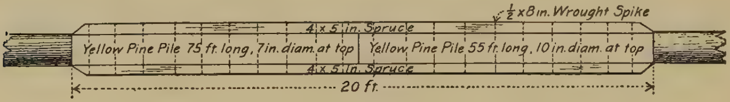

½ x 8 in. Wrought Spike
4 x 5 in. Spruce
Yellow Pine Pile 75 ft. long, 7 in. diam. at top
Yellow Pine Pile 55 ft. long, 10 in. diam. at top
4 x 5 in. Spruce
20 ft.

Fig. 25*f*.—Fish-plate Joint in Long Pile.

a plain fish-plate splice of unusual length used in forming piles 130 ft. long, to support certain parts of the falsework of the Poughkeepsie bridge and in connection with the anchorage of the cribs during the sinking process. An enlarged section is shown in Fig. 25*e*.

Problem 25. Compute the strength of the joint at the center of the lower chord in Fig. 72*c*, assuming unit-stresses per square inch for the wood of 1300 lb. for tension, 1100 and 275 lb. for compression on the ends and sides of the fibers, respectively, and 130 lb. for shear parallel to the fiber; and 15,000 lb. for the bolts in flexure.

Art. 26. Design of Plain Fish-plate Joint

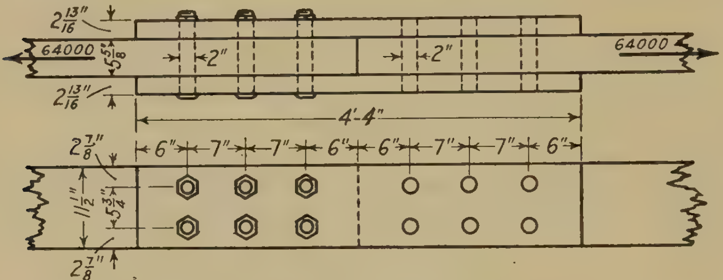
The following order is suggested for the computations needed to make the design: 1. Compute the required net area in tension, the area in compression for square bolts, the equivalent compressive area for round bolts, the shearing area, and the longitudinal area to resist splitting. 2. Assume the depth, observing that the bolts make a large difference between the gross and net width of the cross-section. 3. Compute the bending moment to be resisted by the bolts in one-half of the joint. 4. Assume the number of bolts, the number being even when two rows of bolts and wooden fish plates are used. 5. Determine the diameter of the bolts, and check the diameter by means of the required compressive area. 6. Find the net and gross widths of the timber. 7. Compute the longitudinal distance between centers of bolts, the distance from center of bolt to end of timber, and the total length of the joint. 8. Try a different depth to learn whether better results can be secured.

For example, let the joint be designed to resist the same tension as the tabled fish-plate joint of Art. 21, using the same kind of wood. Besides the safe unit stresses given in that article, the following are required; for tension across the grain of the timber, 150 lb. per sq. in., and for the stress in the outer fiber of the bolt or pin under flexure, 24,000 lb. per sq. in.

In this example the ratio between the allowable compression on the side of the fiber to that on the end of the fiber is $\frac{380}{1713} = 0.222$. The net tensile area is $\frac{64{,}000}{1750} = 36.5$ sq. in.; and compressive area for square bolts $\frac{64{,}000}{1713} = 37.4$; and for round bolts $\frac{37.4}{0.57} = 65.6$ sq. in.; the shearing area is $\frac{64{,}000}{256} = 250$ sq. in.; and the splitting area $\frac{0.095 \times 64{,}000}{150} = 40.5$ sq. in.

Assuming a dressed depth of $5\frac{5}{8}$ in., the bending moment is $32{,}000 \times 2.81 = 89{,}900$ in.-lb. If 6 bolts are used, the resisting moment of each bolt is 14,970 sq. in., which, according to a table of bending moments on pins, requires a diameter of 2 in., the safe bending moment of which is 18,800 in.-lb. The compressive area is then $6 \times 2 \times 5.62 = 67.4$ sq. in., indicating that the number of bolts was correctly assumed. The required net width of the cross section is $\frac{36.5}{5.62} = 6.49$ in., and the required gross width $6.49 + 2 \times 2 = 10.49$. A dressed thickness of $11\frac{1}{2}$ in. will be adopted, the nominal size of the timber being 6 × 12 in. The length of the two shearing surfaces behind each bolt is $\frac{250}{2 \times 6 \times 5.62} = 3.71$ in., and of the splitting surface, $\frac{40.5}{6 \times 5.62} =$ 1.20 in. The longitudinal spacing of the bolts is therefore $3.71 + 1.20 + 2 = 6.91$ or 7 in. between centers, while the distance from center of bolt to end of timber is 1 in. less, or 5 in. The length of the fish plate is $2(2 \times 7 + 2 \times 6) = 52$ in.

For very large stresses the smallest number of bolts which gives the necessary compressive area may require them to be spaced so far apart longitudinally that the transverse tension cannot be properly distributed. It must be remembered that the components of the pressure on each bolt which tends to split the timber is acting at the bolt, and therefore the longitudinal surface resisting this tendency should not extend too far beyond the bolt. It may be better in such cases to increase the number of bolts in order to reduce the combined length of the splitting and shearing surface behind each bolt.


2 13/16"
64000
5 5/8
2"
2"
64000
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4'-4"
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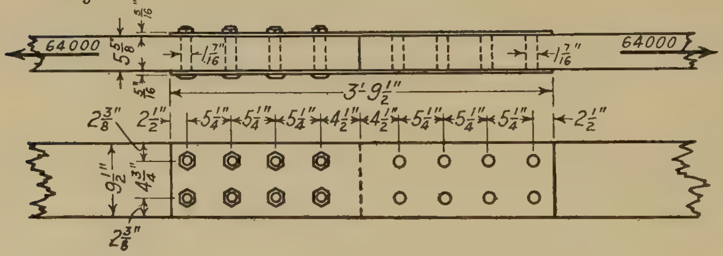
11 1/2"
5 3/4"
2 7/8"


5/16
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5 1/4"
5 1/4"
2 1/2"
9 1/2"
4 3/4"
2 3/8"

FIG. 26a.—Working Drawing of Plain Fish-plate Joint.
FIG. 26b.—Working Drawing of Joint with Steel Fish Plates.

The detail drawing for the above design with all the dimensions required for its construction is given in Fig. 26a. The bill of material for the joint and main timbers, assumed to be 20 ft. long, is as follows:

2 main timbers, 6 × 12 in. × 20 ft. 0 in.	240.0 ft. b.m.
2 fish plates, 3 × 12 in. × 3 ft. 8 in. (4 ft. 0 in.)	36.0 ft. b.m.
12 lateral pins, 2 in. in diam., 13½ in. long, with nuts	160 lb.

The estimate of cost is as follows:

276.0 ft. b.m. lumber @ 6 cents	$16.56
160 lb. lateral pins @ 6 cents	9.60
3 hours' labor @ 80 cents	2.40
Total cost	$28.56

ART. 27. PLAIN FISH PLATES OF STEEL

The use of steel fish plates reduces the length and diameter of the bolts, since the bending moment is less than with wooden fish plates. In designing the fish plate the thickness must be sufficient to provide the necessary bearing area on the bolts. The width is determined by the net tensile area, but as it cannot exceed the width of the timber, the thickness may have to be increased. The plate must be extended far enough beyond each end bolt to act as a beam in order to transfer the pressure or load from the bolt to the net tensile sections on each side of the bolt.

As indicated in Fig. 27*a* an odd number of bolts may be used in each half of the joint, but the longitudinal spacing should be the same as if the number were even. In this case with five bolts, it will be observed that for the timber the critical section in tension is not at the end bolt but at the next pair of bolts, since that section transmits four-fifths of the entire stress in the joint.

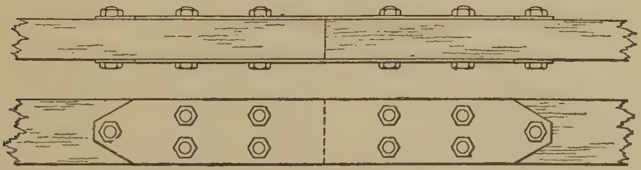


FIG. 27*a*.—Joint with Plain Fish Plates of Steel.

Let a design be made for a plain fish-plate joint carrying the same stress as that in Art. 26 but with fish plates of steel instead of wood. Accordingly the tensile, compressive, shearing and splitting areas for the timber remain the same as before; namely, 36.5, 65.6, 250 and 40.5 sq. in., respectively. The unit stresses for the steel fish plates are to be the same as those used in Art. 23.

Let the depth of the timber be again taken as $5\frac{5}{8}$ in. and let the thickness of the fish plates be provisionally assumed as $\frac{3}{8}$ in. The bending moment to be resisted by all the bolts in each timber is $32{,}000\ (\frac{1}{2} \times 0.375 + \frac{1}{4} \times 5.625) = 51{,}000$ in.-lb. If 8 bolts are assumed, the resisting moment for each bolt is 6375 in.-lb., thus requiring a diameter of $1\frac{7}{16}$ in., the resisting moment of which is 7000 in.-lb. The finished size must be used on account of its bearing on a steel fish plate. The compressive area furnished is $8 \times 5.62 \times 1.437 = 64.7$ sq. in. As this area is only 1.4 per cent less than the required area, 8 bolts will be used.

The net width of the timber cross-section is $\dfrac{36.5}{5.62} = 6.49$ in. and the gross width $6.49 + 2 \times 1.438 = 9.36$, say $9\frac{1}{2}$ in. The net tensile section required for each fish plate is $\dfrac{32{,}000}{16{,}000} = 2.0$ sq. in. The net width of the plate is $9.5 - 2 \times 1.469 = 6.56$ in., the diameter of the hole being $\frac{1}{32}$ in. larger than that of the bolt. The required thickness of the plate is $2.0/6.56 = 0.305$ in. The thickness to be adopted is therefore $\frac{5}{16}$ instead of $\frac{3}{8}$ in. The revised bending moment in each bolt then becomes 6250 in.-lb., and hence the diameter remains unchanged.

The length of the shearing surface behind each bolt is $250/(2 \times 8 \times 5.62) = 2.78$ in., and of the splitting surface $40.5/(8 \times 5.62) = 0.90$ in. The longitudinal spacing of the bolts between centers is $2.78 + 0.90 + 1.438 = 5.12$, or $5\frac{1}{4}$ in., while the distance from center of bolt to end of timber is 4.40, or $4\frac{1}{2}$ in.

Since that part of the tensile stress due to the bearing of one of the end bolts is distributed over a width of 1.641 in. on each side of the bolt, the metal beyond the bolt should be extended far enough to enable it to act as a beam. The bending moment diagram for this beam is similar to that of a bolt in a plain fish-plate joint (see Fig. 25*b*) and hence the maximum bending moment, which occurs at the middle, is $2000\ (\frac{1}{2} \times 1.641 + \frac{1}{4} \times 1.46) = 2370$ in.-lb. Placing this equal to the resisting moment of a beam of rectangular cross-section, having a width of $\frac{5}{16}$ in. and an allowable unit stress in the outer fiber of 16,000 lb. per sq. in., the depth is found to be 1.687 in. The distance from the center of bolt to end of fish plate is then $1.687 + \frac{1}{2} \times 1.438 = 2.406$, or $2\frac{1}{2}$ in. The total length of each fish plate is $2(3 \times 5.25 + 4.5 + 2.5) = 45\frac{1}{2}$ in.

The detail drawing, with all the required dimensions, is given in Fig. 26*b*. The bill of material is as follows:

2 timbers 6 × 10 in. × 20 ft. 0 in.	200.0 ft. b.m.
2 steel plates $9\frac{1}{2} \times \frac{5}{16}$ in. × 3 ft. $9\frac{1}{2}$ in.	72.5 lb.
16 lateral pins $1\frac{7}{16}$ in. in diam., 7 in. long, with nuts	67 lb.

The estimate of cost is as follows:

200 ft. b.m. lumber @ 6 cents	$12.00
72.5 lb. steel plates @ 6 cents	4.35
67 lb. lateral pins @ 6 cents	4.02
3 hrs. labor @ 80 cents	2.40
	$22.77

A comparison of the designs of the four types of fish-plate joints may now be made, as follows:

1. Tabled fish-plate joint with wooden plates	$26.66
2. Tabled fish-plate joint with steel plates	28.89
3. Plain fish-plate joint with wooden plates	26.40
4. Plain fish-plate joint with steel plates	22.75

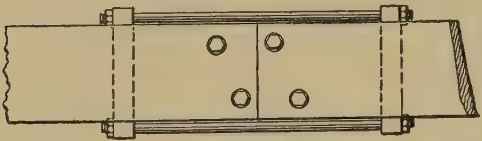
ART. 28. TENON-BAR SPLICE

As shown in Fig. 28*a* the tenon-bar splice consists of one or more bars of steel acting as a beam on each side of the joint, the corresponding ends of both bars being connected with ordinary bolts. Since the ends of the fibers bear on plain surfaces which are perpendicular to them, less width is required for the bearing area of the wood; and as there is no tendency to split the timber, the shearing surfaces are shorter than for round bolt beams. The tenon bar may be made wide enough to resist its bending moment or to keep its deflection within any assigned limit, so that the pressure of the fibers may be regarded as an uniformly distributed load. The thickness of the bar must provide sufficient strength outside of the bolt holes.

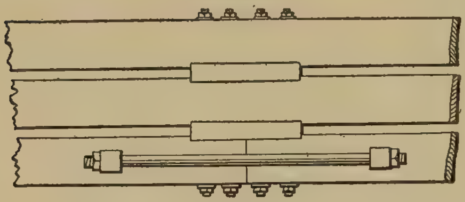
When used in splicing narrow timbers or planks the bars are connected by U-bolts, since their thickness is not sufficient to allow for bolt holes through them.

This type of splice is notable for its simplicity, directness of of action and certainty in the computation of its working value. The carpenter work is comparatively small and of such a nature that defects are not so liable to occur through careless workmanship as for other types of joints. Another advantage is that loosening due to shrinkage can be overcome by tightening the bolts at any time.

In designing this type of splice there must be determined (1) the size of the bolt, (2) width of bar for bearing, (3) depth of bar for bending, (4) net section of timber, and (5) distance of bar from end to provide sufficient shearing area.



Elevation.



Plan.

FIG. 28*a*.—Tenon-bar Splice.

For example, let the joint be designed to resist the same tension as the tabled fish-plate joint of Art. 22, using the same kind of wood. Let four bolts be assumed. The necessary tensile area at the root of thread for each bolt will be $\frac{64,000}{4 \times 16,000} = 1$ sq. in. A $1\frac{3}{8}$-in. bolt will be used, the area at the root of thread being 1.05 sq. in. The long diameter of a $1\frac{3}{8}$-in. hexagonal nut is $2\frac{9}{16}$ in., hence the distance from the side of the timber to the center line of the bolt will be made $1\frac{5}{16}$ in. The required bearing area of bar is $\frac{64,000}{2 \times 1713} = 18.7$ sq. in. If a $7\frac{5}{8}$- by $11\frac{1}{2}$-in. section is assumed, the required width of bar is $\frac{18.7}{7.62} = 2.45$ in. In order that the nut shall have full bearing, make this width $2\frac{3}{4}$ in. The bending moment on the bar is, assuming a uniform distribution of load,

$$16,000\ (1.31 + \tfrac{1}{4} \times 7.62) = 51,520 \text{ in.-lb.}$$

The depth of bar, d, is given by the formula $\frac{1}{6} \times 16{,}000 \times 2.75d^2 = 51{,}520$, or $d = 2.83$ in. Hence the bar will be made $2\frac{3}{4}$ by 3 by 14 in. The net tensile area required is $\frac{64{,}000}{1750} = 36.6$ sq. in. The area furnished is $(11.5 - 2 \times 2.75) \times 7.62 = 43.3$ sq. in. The shearing area required for one bar is $32{,}000/256 = 125$ sq. in. The distance required between the bar and end of timber is $125/7.62 = 15.1$ in., say 16 in. Figure 28*b* shows the drawing for this design. Splice pads are used to give general stiffness to the joint. The bill of material is as follows:

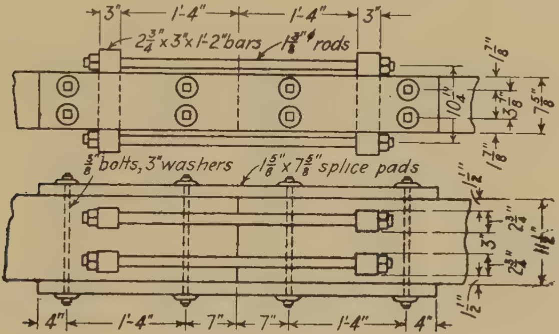


FIG. 28*b*.

2 timbers 8 × 12 in. × 20 ft. 0 in.	320 ft. b.m.
2 splice pads, 2 × 8 in. × 4 ft. 6 in.	12 ft. b.m.
4 steel plates, $2\frac{3}{4}$ × 3 in. × 1 ft. 2 in.	131 lb.
4 $1\frac{3}{8}$-in. bolts, 3 ft. 5 in. long; with nuts	80 lb.
8 $\frac{5}{8}$-in. bolts, 12 in. long, with nuts	10 lb.
16 ogee washers, 3 in. in diam., @ .75 lb.	12 lb.

The estimate of cost is as follows:

332 ft. b.m. lumber @ 6 cents	$19.92
131 lb. steel plates @ 4 cents	5.24
10 lb. bolts @ 5 cents	.50
12 lb. washers @ 5 cents	.60
4 hrs. labor @ 80 cents	3.20
Total cost	$29.46

ART. 29. SHEAR-PIN SPLICE

In making a shear-pin splice two pads are spiked to the sides of the timber to be spliced as shown in Fig. 30*a*. Holes are bored and then bolts are driven into these holes and the nuts screwed up. Holes for the circular pins are then bored and the pins driven. This type of splice is easily made and is inexpensive, but it is suitable only with well seasoned timber, since any shrinkage will allow slipping.

In designing this type of joint the following elements must be considered: tension in the main timber and splice pads, bearing by the pins on the timber and splice pads, shearing and distortion of the pins, and tension in the bolts.

Various materials have been used for the pins, but tests made at the University of California[1] indicate that for splicing Douglas fir, ordinary gas pipe and oak pins will not develop the compressive strength of the timber, the pipe distorting and the oak pins shearing. On the other hand, extra heavy pipe, solid steel pins and Hawaiian Ohia wood pins develop the full strength of the timber. The tests gave the following results in pounds per linear inch of pin, all joints tested having four pins, each 6 in. long, the timber being 6 by 6 in. in section and the splice pads 3 by 6 in.

	$1\frac{1}{2}$-in. Gas Pipe	2-in. Gas Pipe	2-in. Oak	2-in. Extra Heavy Pipe	2-in. Solid Steel Pin	2-in. Ohia Wood
At slip of about $\frac{1}{16}$ in.	1083	1540	1112	1880	1995	1875
At ultimate load	1943	2220	2175	2480	2710	2790

A slip of $\frac{1}{16}$ in. seemed to represent approximately the elastic limit of the joints, or the point at which equal increments of load lead to increasing increments of slip. This agrees with the results noted in Art. 6 on the lateral strength of nailed joints. On the basis of the above tests, 800 lb. per linear inch of pin was suggested as a safe design value for 2-in. extra heavy pipe on Douglas fir.

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[1] Eng. News, vol. 71, p. 593, March 19, 1914.

In proportioning the joints for these tests the bolts were designed for a total tension equal to one-half the load on the joint and the results seem to indicate that this was about right. This value is much higher than those given in Art. 24 due to the fact that here each pin is acted on by the two forces *P′*, which tend to cause a rotation of the pin, this rotation being resisted by the forces *P″* of Fig. 29*a*.

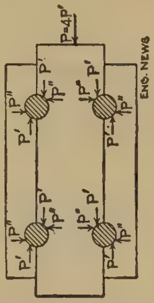


FIG. 29*a*.—Pin Pressure on Pin-keyed Joints.

ART. 30. DESIGN OF SHEAR-PIN SPLICE

Let it be required to design a shear-pin splice to transmit a tension of 64,000 lb. The timber to be used is dense select Coast region Douglas fir and the joint is in a building protected from the weather. Let the working stresses be specified as follows in pounds per square inch: for tension 1750; for compression with the grain 1713; for compression across the grain 380; and for shear along the grain 210. Let the unit tensile stress in the bolts be taken at 16,000 lb. per sq. in. The net required tensile area is $64{,}000 \div 1750 = 36.5$ sq. in., and the net shearing area $64{,}000 \div 210 = 304.5$ sq. in. The ratio between the allowable compression on the side of the fiber to that on the end of the fiber is $380 \div 1713 = 0.222$; hence, by Art. 24, the ratio of round pin strength to square pin strength is 0.57. Using 2-in. extra heavy steel pipe for the pins, the total required length is $\dfrac{64{,}000}{1713 \times 0.57} = 65.6$ in.

Assuming a dressed depth of $7\frac{1}{2}$ in., the required net dressed width is $\dfrac{36.5}{7.5 - 2 \times 1} = 6.64$ in., or $7\frac{1}{2}$ in. Hence a commercial timber 8 by 8 in. will be adopted. The number of pins to be used is $65.6 \div 7.5 = 8.75$, or 10. The length of the shearing surface between pins is $\dfrac{304.5}{7.5 \times 10} = 4.06$. For the longitudinal spacing of the pins, the diameter of the pin will be added to this for the reasons given in Art. 25. Hence the longitudinal spacing of pins is 4.06 + 2 = 6.06, or $6\frac{1}{2}$ in., to provide for shearing area taken away by the bolt holes.

Designing the bolts for a stress equal to one-half the tension in the joint and using 12 bolts, the stress in each bolt is 32,000 ÷ 12 = 2666 lb. The required area at the root of the thread is 2666 ÷ 16,000 = 0.167 sq. in. A $\frac{5}{8}$-in. bolt furnishes 0.202 sq. in. The required bearing area of washer is 2665 ÷ 380 = 7.0 sq. in. According to Table 2*c* a $3\frac{1}{8}$-in. ogee washer furnishes an area of 7.23 sq. in.

Each splice pad will be one-half the size of the main timber, since the two splice pads take the load of the main timber.

The detail drawing for this design, with all the dimensions required for its construction, is given in Fig. 30*a*. The bill of material for the joint and the main timbers, which are assumed to be 20 ft. long, is as follows:

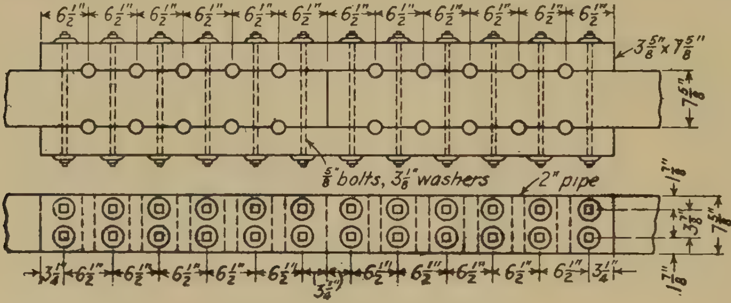


FIG. 30*a*.—Shear-pin Splice.

2 main timbers, 8 × 8 in. × 20 ft. 0 in.	213.3 ft. b.m.
2 splice timbers, 4 × 8 in. × 6 ft. 6 in. (8 ft. 0 in.)	42.7 ft. b.m.
20 extra strong, 2-in. pipe, $7\frac{1}{2}$-in. long	62.5 lb.
24 $\frac{5}{8}$-in. bolts, $17\frac{1}{2}$-in. long; 12 nuts	41.8 lb.
48 ogee washers, $3\frac{1}{8}$ in. in diam., @ 0.562 lb.	27.0 lb.

The estimate of cost is as follows:

256.0 ft. b.m. lumber @ 6 cents	$15.36
62.5 lb. 2-in. pipe @ 5 cents	3.12
41.8 lb. bolts and nuts @ 5 cents	2.09
27 lb. washers @ 6 cents	1.62
4 hrs. labor @ 80 cents	3.20
Total cost	$25.39

Art. 31. Lap and Scarf Joints

In a lap joint one piece laps over the other at the connection. There are two general forms of the plain lap in use, one where the edges overlap as in a common form of weather boarding, and the other where the sides overlap as in Fig. 31*a* and are bolted together. Sometimes keys are inserted between the sticks to resist the longitudinal shear in the joint. This joint should not be used to transmit either tension or compression unless these stresses are small and merely incidental to the main function of the timbers spliced. The bending moment in the bolts is large in proportion to the longitudinal stress, and the tendency is to crush the fibers against the bolts near the joint surface as indicated in Fig. 31*b*. The lap joint is not infrequently used to lengthen the booms or back stays of a derrick, for which purpose the plain fish-plate joint is better adapted.

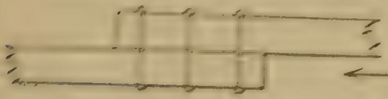


Fig. 31*a*.—Lap Joint.

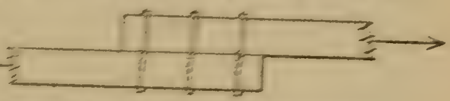


Fig. 31*b*.—Lap Joint under Tension.

A scarf joint is made by cutting away opposite sides from the ends of two timbers so that they may be lapped to form a continuous piece without increased thickness. There is a variety of scarfs depending upon the character of the cutting on each timber, some of which are ingeniously devised. In general the simpler forms are the best, since they can be more accurately fitted together and at less cost for labor.

The simplest scarf is that in which one-half of the thickness of each end is cut away as in Fig. 31*c*, and is therefore called the half lap. When used to splice compression members, its length is made about twice the depth of the timber and the bolts are placed at the quarter points. Its application to splicing guard rails on bridges is illustrated in Fig. 31*d*.

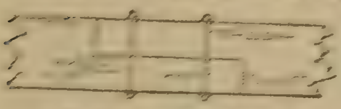


Fig. 31*c*.—Half Lap.

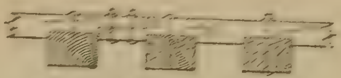


Fig. 31*d*.—Splice in Guard Timber.

Figure 31*e* shows the addition of keys to a half-lap joint which is intended to resist some tension. This form has been used in splicing the lower chords of wooden roof trusses in some exposition buildings, but it is not advisable to employ it when the tension to be transmitted equals a large percentage of the net strength of the timbers to be spliced. It is also used in framing composite and laced posts (Art. 53).

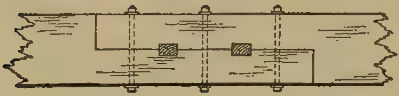


Fig. 31*e*.—Half Lap with Keys.

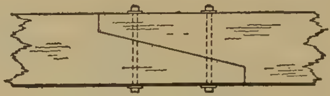


Fig. 31*f*.—Oblique Scarf.

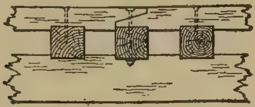


Fig. 31*g*.—Splice in Guard Timber.

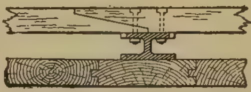


Fig. 31*h*.—Splice in Running Board on Freight Car.

The oblique scarf is shown in Fig. 31*f*. It is customary to cut down each end to one-fourth of the depth of the stick and to make the length of the joint from two to three times the depth. It will be noted that this form has more than double the flexural strength of the half lap. Figure 31*g* indicates its application to the splice of a guard rail and Fig. 31*h* to that of a running board on the top of a freight car. Figure 31*o* shows the framing of a guyed pile driver in which the leads are spliced by oblique scarfs. It was built by James H. Fuertes for use in driving trestle piles across very steep valleys on country roads. This form of scarf is also used in car sills other than the center sills to which the draw timbers are attached. When the inclined surface slopes in the opposite direction as in Fig. 31*i*, the joint is called beveled halving.

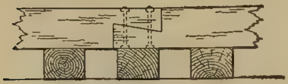


Fig. 31*i*.—Beveled Halving.

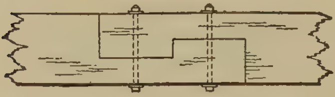


Fig. 31*j*.—Straight Tabled Scarf.

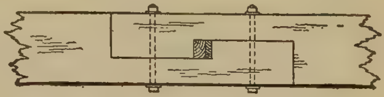


Fig. 31*k*.—Tabled Scarf with Key.

The straight tabled scarf is illustrated in Fig. 31*j* and with the addition of a key in Fig. 31*k*. This joint can transmit some tension as well as compression, and when the magnitude of the stress is known, the required dimensions of the parts can be readily computed. When the stress cannot be determined, the length is made about 2½ times the depth of the timber, whereas height of the table is one-fourth of the total depth.

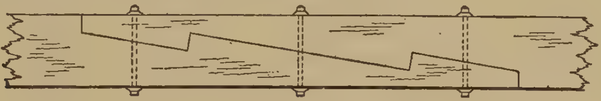


FIG. 31*l*.—Oblique Tabled Scarf.

The oblique tabled scarf has two or more sloping surfaces of contact with an offset or shoulder between them, thus forming inclined tables. Figure 31*l* shows such a joint with two tables on each timber. The height of the table equals that of the neck, which in this case equals one-fourth of the depth of the timber. The net section available for tension equals 50 percent of the gross section of the stick. The eccentric application of the stress tends to develop considerable secondary bending in each part of the joint as well as in the splice as a whole. The great length required to secure sufficient shearing surface parallel to the fibers makes it an uneconomical form of joint. When there is but one table on each timber, the available net section is 33 percent. It will be observed that the bolts are not located in the sections containing the net depth as is the case with the straight tabled scarf.

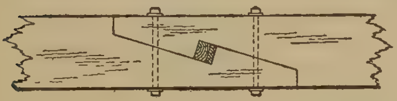
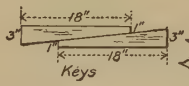


FIG. 31*m*.—Oblique Tabled Scarf with Key.



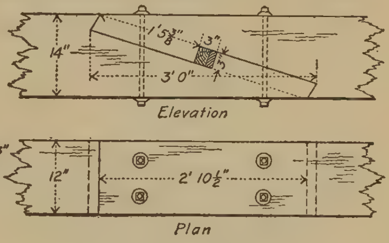


FIG. 31*n*.—Splice in Sill of Framed Trestle.

Figure 31*m* shows the same form with the addition of a key. In practice the depth of the table is often made about one-fourth of the depth of the timber, while the length of the joint varies from 2½ to 3½ times the total depth. Figure 31*n* gives the form and dimensions of a splice in the sill of a high trestle bent. A simpler form of joint would probably be used at present.

An important series of drop tests was made in 1908 by a Committee of the Master Car Builders' Association to determine the relative value of the half-lap and oblique-scarf joints in splicing the sills of freight cars. The impact was applied on the ends of spliced members 8 ft. long, subjecting them to longitudinal compression, thus representing conditions which cause the largest number of failures of sills in service. The center or draft sills have a fish plate called a "liner" added on one side of the splice to give it additional strength and stiffness. The sills were either of yellow pine or Douglas fir, and the liners were of oak or yellow pine. By comparing the average height of drop which caused failure for each type of joint, it was found that the combination of scarf and liner had a resistance of 23.5 percent of that of the straight sill, whereas the combination of the half-lap and liner had a corresponding resistance of 88 percent. The splices were also subjected to comparative transverse and tensile tests and in both cases proved the superiority of the half-lap over the scarf joint. For the detailed record of data and tests, including deflection and other diagrams see Pro-

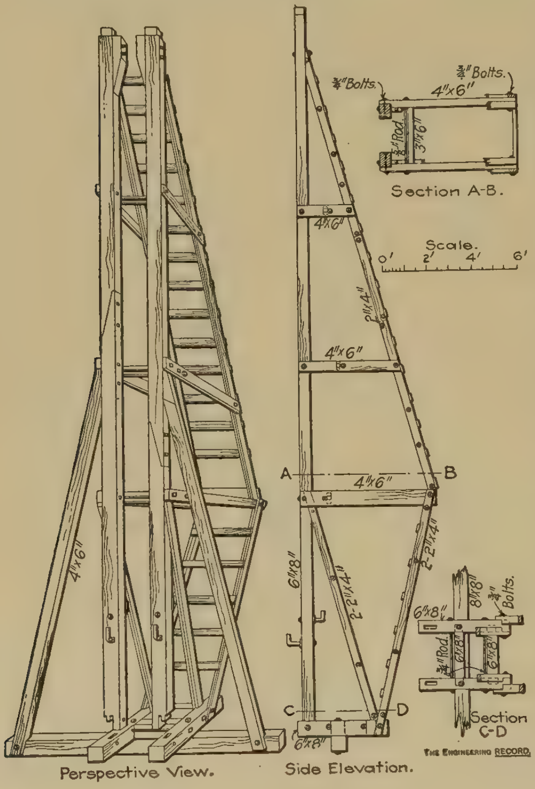
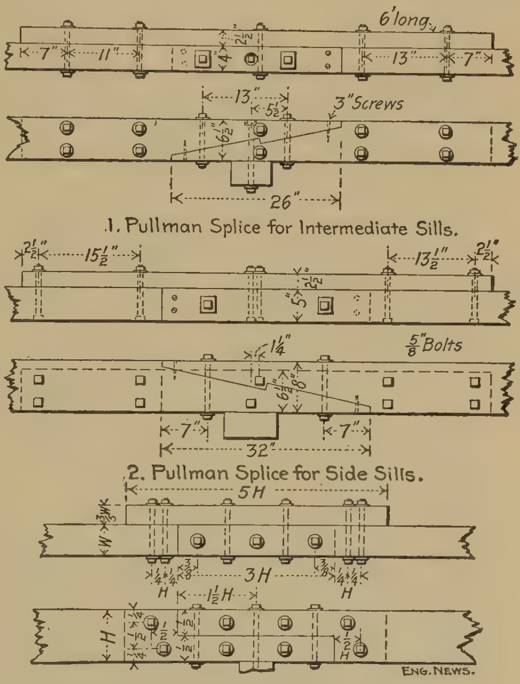

¾" Bolts.
¾" Bolts.
4"x6"
¾" Rod
3"x6"
Section A-B.
Scale.
0' 2' 4' 6'
4"x6"
2-2"x4"
4"x6"
A
B
4"x6"
2-2"x4"
6"x8"
2-2"x4"
4"x6"
6"x8"
8"x8"
¾" Bolts
¾" Rod
6"x8"
6"x8"
C
D
6"x8"
Section C-D
The Engineering Record
Perspective View.
Side Elevation.

Fig. 31*o*.—A Guyed Pile Driver.

ceedings of the Master Car Builders' Association, 1909, vol. 43, page 227. An abstract of the report of the committee was published in Railroad Age Gazette, vol. 46, page 1489, June 24, 1909.



1. Pullman Splice for Intermediate Sills.

2. Pullman Splice for Side Sills.

3. Splice Standing Highest Test.

FIG. 31*p*.—Splices for Passenger Car Sills.

A committee of the same Association in 1902 had called attention to the relative weakness of various forms of the scarf as compared with the half-lap joint (called step splice by the Committee) in a report on the splicing of passenger car sills. The oblique tabled scarf without key is shown in Fig. 31*p* in the splices for intermediate and side sills; the proportions of the joint recommended for adoption are given below them. A liner is also shown in each splice. The results of 22 compression and transverse tests on four types of splices made for the Committee are given in their report in Proceedings, vol. 36, page 137. The compression tests were without impact, and the methods of failure are shown by halftone illustrations. The report shows all the kinds of car sill splices in use at that time.

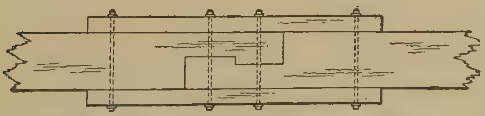


FIG. 31*q*.—Splice in Sill of Trestle Bent.

An examination of framing plans published in engineering periodicals and in books on building construction reveals a large variety of designs for splices in which two different types of joints are combined, both intended to perform practically the same functions. Some of these combinations are as follows: Plain fish plate and half lap (Fig. 32*k*); plain fish plate and straight tabled scarf (Fig. 31*q*); plain fish plate and oblique scarf with keys (Fig. 31*r*); tabled fish plate on one side and plain fish plate on the other; plain and tabled fish-plate joint with steel plates (Fig. 72*g*).

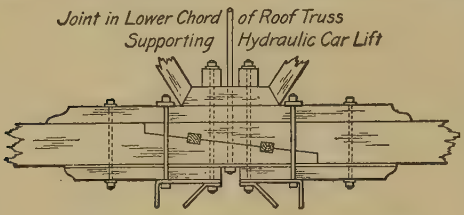

Joint in Lower Chord of Roof Truss
Supporting Hydraulic Car Lift

FIG. 31*r*.—Composite Type of Joint.

ART. 32. VARIOUS JOINTS IN BEARING

A butt joint is one in which the end or edge of a timber comes squarely against another piece (Fig. 32*a*). This is the most effective joint between the ends of two compression members, the ends being held in place by other connecting braces as in Fig. 53*b*. In Fig. 32*b* the horizontal rails meet each other in a butt joint at their connection with a vertical post, but transmit little, if any, stress from one to the other. The use of iron straps and plates to prevent lateral displacement in the butt joint is described in Art. 19.

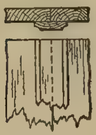


FIG. 32*a*.—Butt Joint in Weather Boarding.

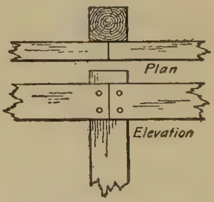


FIG. 32*b*.—Butt Joint in Rail.

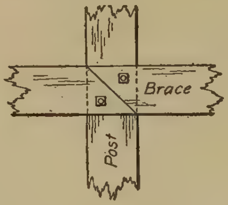


FIG. 32*c*.—Bevel Joint in Brace.

A bevel is any inclination of a cross-section other than a right angle. In Fig. 32*c* both braces are cut on a bevel, but in opposite directions.

A miter joint is made by beveling the pieces joined so that the plane of the joint bisects the angle between them, as at the corner of a picture frame. The application of this joint to tunnel timbering is shown in Figs. 32*d* and *e*. Unless the timber is well seasoned the transverse shrinkage of the wood causes the miter joint to open on the inner side and thus concentrate the compression on the outer edges. Another application is given in Fig. 32*f* between the planks of which the rib is built up.

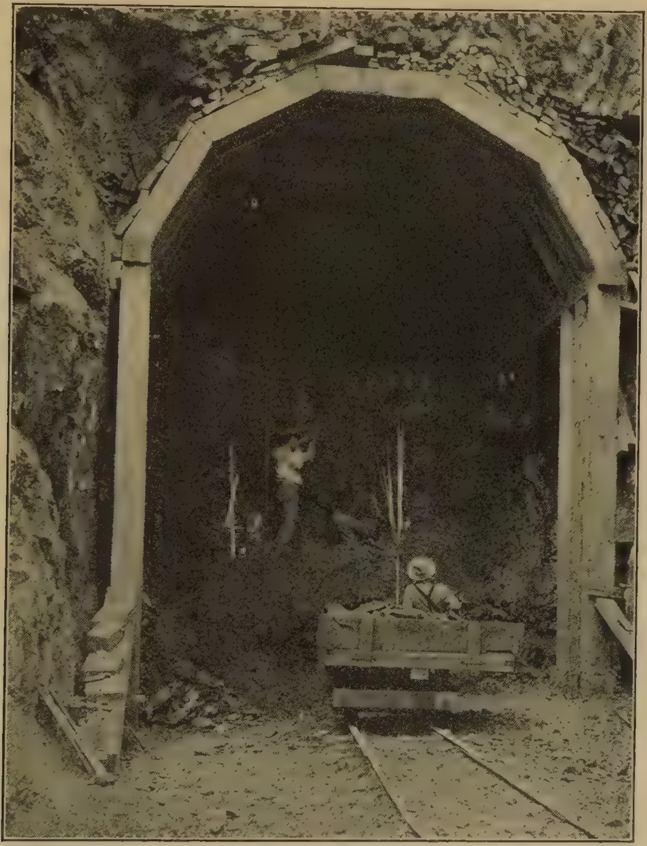
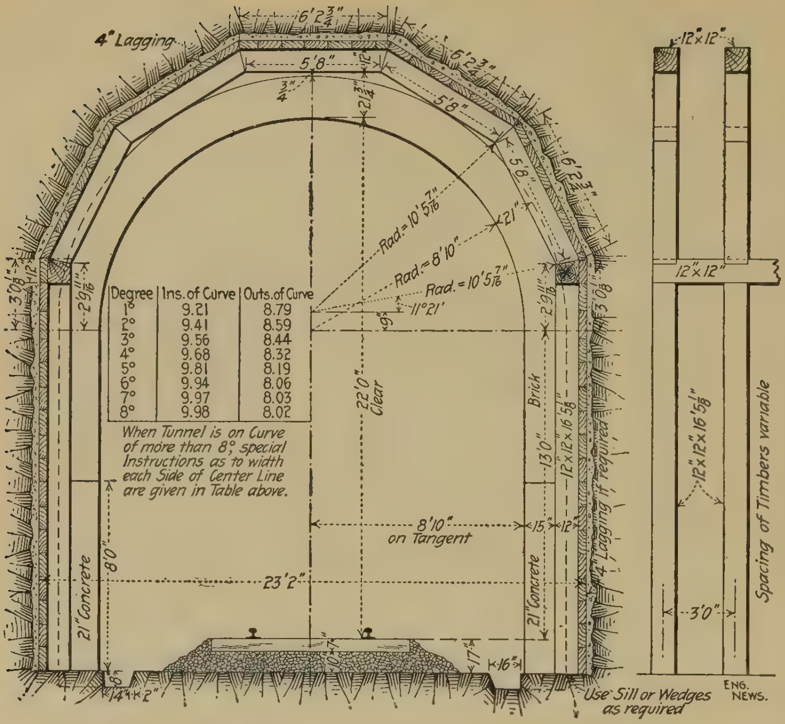


FIG. 32*d*.—Segmental Timbering, Used in Tunnel Work on the Western Pacific Railway. See *Railroad Age Gazette*, vol. 45, page 902, Sept. 11, 1908.

This joint is often used with some modifications at the hip of a truss as illustrated in Figs. 32*g* and *h*. The former is from a temporary truss for contractors' use, and the latter from a roof truss. In the latter case two washers are used in order to secure

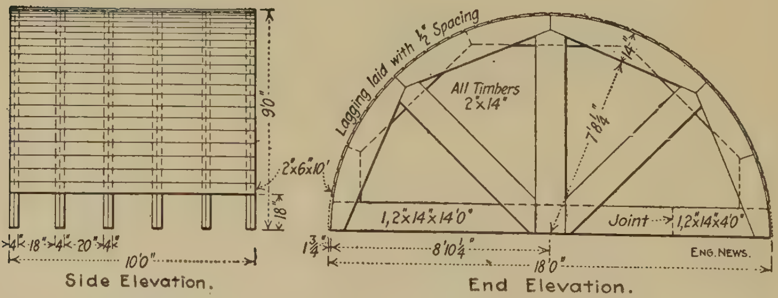


Degree	Ins. of Curve	Outs. of Curve
1°	9.21	8.79
2°	9.41	8.59
3°	9.56	8.44
4°	9.68	8.32
5°	9.81	8.19
6°	9.94	8.06
7°	9.97	8.03
8°	9.98	8.02

When Tunnel is on Curve of more than 8°, special Instructions as to width each Side of Center Line are given in Table above.

4" Lagging
6'2¾"
5'8"
12"
¾"
2¼"
6'2¾"
5'8"
5'8"
6'2¾"
Rad.=10'5 7/16"
Rad.=8'10"
Rad.=10'5 7/16"
21"
11°21'
9"
3'0⅝"
12"
2'9 11/16"
2'9 11/16"
3'0⅝"
22'0" Clear
Brick
13'0"
12"x12"x16'5⅝"
4" Lagging if required
8'10" on Tangent
15"
12"
21" Concrete
8'0"
23'2"
21" Concrete
10"
17½"
16"
8"
14"
2"
Use Sill or Wedges as required
12"x12"
12"x12"
12"x12"x16'5⅝"
3'0"
Spacing of Timbers variable
ENG. NEWS.

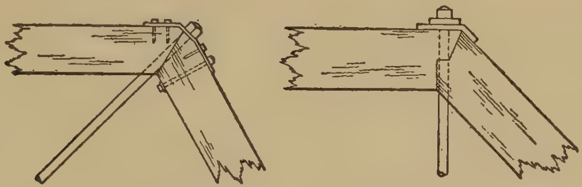
FIG. 32*e*.—Timbering and Concrete Lining of the Langville Tunnel on the Kanawha and Michigan Railway.



9'0"
2"x6"x10'
18"
4" 18" 4" 20" 4"
10'0"
Side Elevation.
Lagging laid with ½" Spacing
All Timbers 2"x14"
14"
7'8¼"
1,2"x14"x14'0"
Joint
1,2"x14"x4'0"
1¾"
8'10¼"
18'0"
ENG. NEWS.
End Elevation.

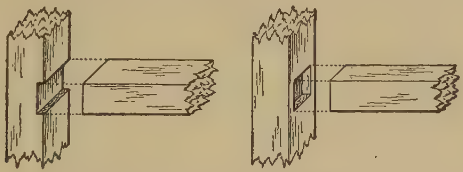
FIG. 32*f*.—Arch Centering Used in the Langville Tunnel. See *Eng. News*, vol. 61, page 11, Jan. 7, 1909.

a larger bearing area without excessive thickness in the plate from which they are cut.



FIGS. 32*g* and *h*.—Joints at Hips of Trusses.

Housing consists in letting the whole end of one timber for a slight distance into the side of another. The term "housing" is also applied to the groove or recess cut into one piece into which the other piece is said to be housed (Fig. 32*i* and *j*). In Fig. 32*k* the horizontal strut is let into a triangular groove in the post, and may be called beveled housing.



FIGS. 32*i* and *j*.—Housing.

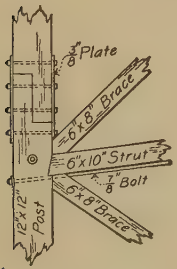

$\frac{3}{8}$" Plate
6"x8" Brace
6"x10" Strut
$\frac{7}{8}$" Bolt
6"x8" Brace
12"x12" Post

FIG. 32*k*.—Intermediate Joint in Tower.

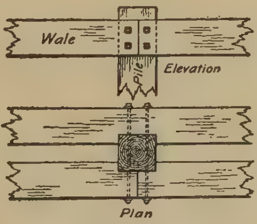

Wale
Pile
Elevation
Plan

FIG. 32*l*.—Single Notching.

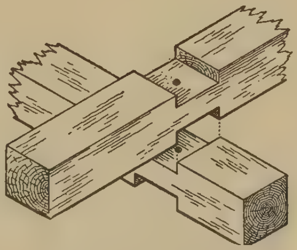
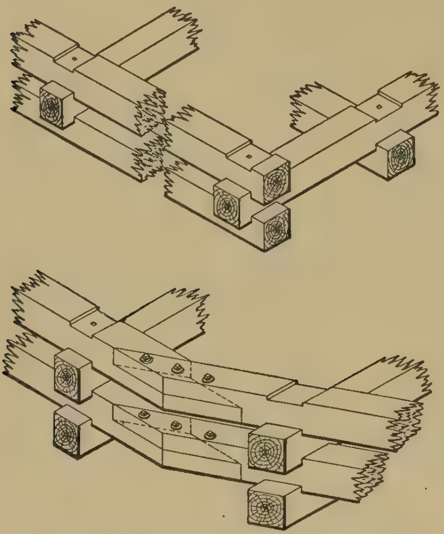


FIG. 32*m*.—Double Notching.

Notching consists in cutting a groove in the side of one timber to insert the side of another. When only one of the timbers is notched, it is called single notching (Fig. 32*l*); and when both timbers are notched, it is called double notching (Figs. 32*m*, *n*, *o*, and *p*). The term "gaining" is often used instead of notching, especially in car construction. It is said that the timbers are "boxed out" in order to "gain" them into each other. In trestle construction the notching is often called "dapping."



Figs. 32*n* and *o*.—Details of Crib Framing.

No form of joint is probably used so extensively as notching in all kinds of timber structures, but especially in the framing of cribs for foundations, docks, breakwaters, etc. Figures 32*n* and *o* show the details of ballasted cribs used in New York City in municipal dock construction. See Engineering Record, vol. 58, page 525, November 7, 1908. The economic value and efficiency of notching in the construction of a frame to support heavy loads was perhaps never so well shown as in the falsework described in Engineering News, vol. 29, page 223, March 9, 1893. Figure 32*q*, which is reproduced from that article, indicates such an arrangement of the truss that only two tension members were required in each half arch, thus simplifying the framing of the joints. The deck timbers were arranged to bring the loads only to the panel points. The only iron used in the falsework con-

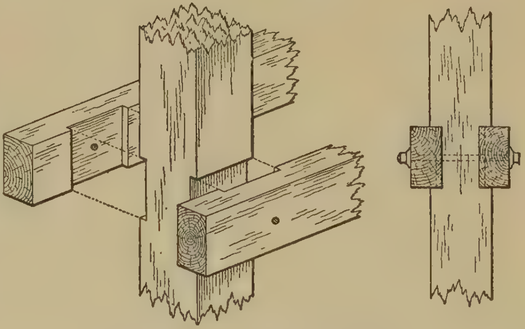


Fig. 32*p*.—Double Notching.

sisted of boat spikes ½ in. square and 12 in. long. The article referred to describes further details and the difficulties encountered in the work owing to the location, and gives additional illustrations.

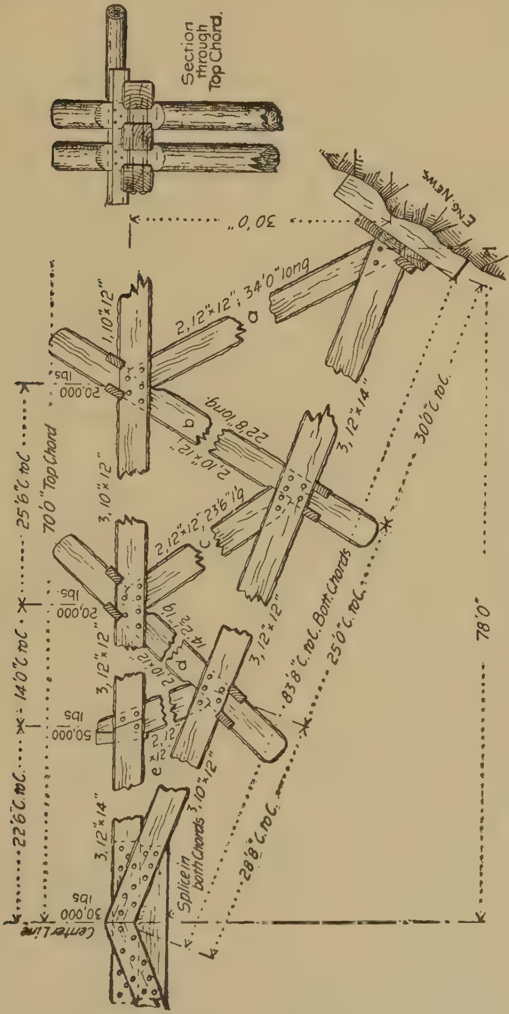
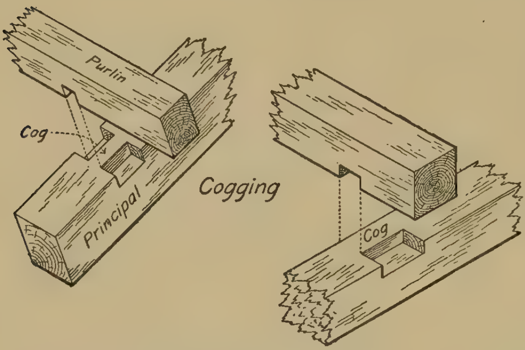

Section through Top Chord.
Eng. News
30'0"
2, 12"x12", 34'0" long
a
1, 10"x12"
20,000 lbs.
b
22'8" long
2, 10"x12"
3, 12"x14"
30'0" c. to c.
3, 10"x12"
2, 12"x12", 23'6" lg
c
25'6" to c.
70'0" Top Chord
20,000 lbs.
14'0" c. to c.
83'8" c. to c. Bott. Chords
25'0" c. to c.
78'0"
3, 12"x12"
d
3, 12"x12"
50,000 lbs.
2, 12" e x ½"
3, 10"x12"
22'6" c. to c.
3, 12"x14"
Splice in both Chords
28'8" c. to c.
30,000 lbs.
Center Line

Fig. 32*q*.—Half Arch of Falsework, Showing Details of Framing.

This novel trussed arch falsework was used to erect a steel bridge over Pend d'Oreille river at Albeni Falls on the Great Northern Railway, under unusually difficult circumstances. When the falsework was built, the snow was three feet deep, and the nearest railway point was thirty-five miles away, by trail through the woods. No sawed timber could be obtained except a few planks floated down the river. On account of the dangerous current the falsework had to be built out from the shore, making the connections in the air. Spruce and tamarack were selected for the compression members and a very tough fir for the tension members.

Cogging differs from notching in not having the cut on one of the timbers extending entirely across, as indicated in Figs. 32*r* and *s*. This kind of joint is used much less than formerly, when metal fastenings were not so abundant and cheap.

In halving each timber is cut to one-half of its depth, so that when the two are united the upper and lower surfaces are flush. When the pieces are joined lengthwise, it is a form of scarfing known as the half lap, but when the pieces cross each other it is a form of notching. Halving is a common method of joining wall plates or other timbers at a corner as in Fig. 32*t*. In beveled halving the surface of contact is inclined (Fig. 32*u*).



Figs. 32*r* and *s*.—Cogging.

The bird's-mouth joint consists of an angular notch cut in one piece of timber so as to make it fit obliquely on the side of another. Both forms of this joint are shown in Fig. 32*v*, one on the side elevation and the other on the end elevation of the top of a cableway tower. The most frequent application of this joint is between a rafter and its supporting wall plate. Another application is shown in Fig. 32*d*, at the joint between the lower sticks of the timber arch and the plumb-post caps.

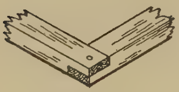


Fig. 32*t*.
Halving.

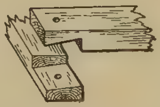


Fig. 32*u*.
Beveled Halving.

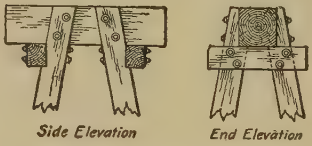
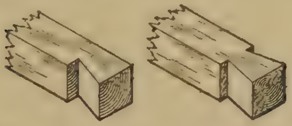


Fig. 32*v*.—Bird's-mouth Joints.

Problem 32. Locate the bird's-mouth joint in Plate V.

Art. 33. Dovetail Joints

A dovetail is a projection having the form of a truncated wedge on the end of one timber, intended to fit into a recess, or groove, of corresponding shape, in the side of another timber, so that the former may not be withdrawn in the direction of its length. When the dovetail is formed by cutting a triangular groove, or notch, on only one side of the timber, it is called a single dovetail (Fig. 33*a*), and when formed by cutting a notch on both sides, it is called a double dovetail (Fig. 33*b*).



FIGS. 33*a* and *b*.—Single and Double Dovetail.

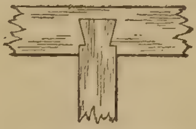


FIG. 33*c*.—Housed Dovetail.

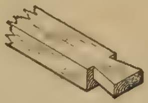


FIG. 33*d*.—Dovetail Halving.

This form of joint is used frequently in cribs for holding broken stone or concrete, and in cofferdams, since the pieces can resist both tension and compression. In Fig. 33*e* the sides of the dovetails are sloped both ways, so that the dovetails on both sides of the corner of crib are alike. The housed dovetail is illustrated in Fig. 33*c*, and dovetail halving in Fig. 33*d*.

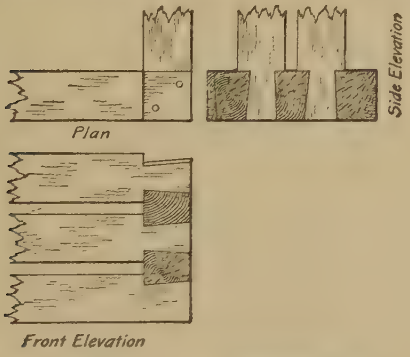


FIG. 33*e*.—Corner of Snowshed Crib.

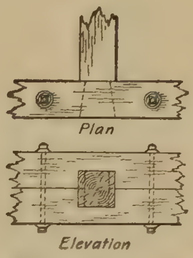


FIG. 33*f*.—Joint for Test.

In 1876 a set of 36 tests was made by C. P. Gilbert, under the direction of General O. M. Poe, Corps of Engineers, U.S.A., to determine the relative holding power of different forms of dovetails used in locking crib walls together by timber ties. The timbers were of undressed white pine, and 6 in. square. The dovetail tie was secured between two pieces of the same section in notches cut out to fit the dovetail as indicated in Fig. 33*f*. The pieces representing the wall were united by bolts placed 6 in. from each side of the tie. The tension in the tie was resisted by reactions on the timber walls 15 in. on each side of the center.

The following conclusions were drawn from the tests: In the strongest form of double dovetail the depth of each notch equals one-sixth of the width of the stick. In the strongest form of single dovetail the depth of notch equals one-third of the width of the stick. The average strength of the double dovetails is greater than that of the single. The strongest form of double dovetail loses less from looseness at the point than at the neck. Smoothing the back of a single dovetail increases its strength. The position of the bolts does not materially affect the strength.

The average strength of the strongest form of double dovetail was found to be 12,915 lb., and that of the single dovetail was 11,875 lb., the difference being about 9 percent. Failure generally occurred in the walls, and only in a few cases did the dovetail shear. The results were originally published in Report of Chief Engineers, U.S.A., 1884, page 2069, and abstracted in Engineering News, vol. 24, page 549, December 20, 1890.

Art. 34. Mortise-and-tenon Joints

In a joint of this type a tenon or rectangular projection on the end of one timber is inserted into a socket or mortise in the side of another stick (Fig. 34*a*). The length of the tenon is made a little shorter than the depth of the mortise so that the shoulders only may bear upon the sides of the other timber; otherwise, the shrinkage of the timber containing the mortise may throw the entire weight on the tenon. The tenon is usually fastened in the mortise by either one or two wooden pins or treenails. The pins

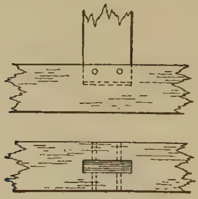


Fig. 34*a*.—Mortise and Tenon.

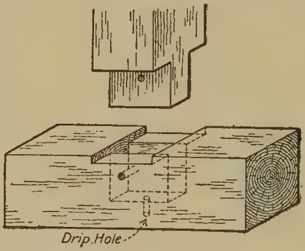


Fig. 34*b*.—Housed Tenon.

are placed at one-third of the length of the tenon from the shoulder. For timbers 10 to 12 in. thick the treenails should be from 1 to 1½ in. in diameter with a taper of ⅛ to ¼ in. The hole in the tenon should be slightly nearer the shoulder than in the mortise, so that when the pin is driven in place it will draw the two timbers into close contact.

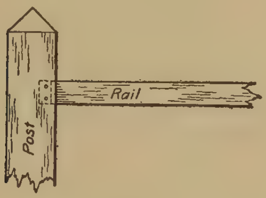
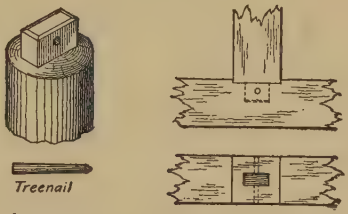


FIG. 34*c*.—Housed Tenon.



FIGS. 34*d* and *e*.—Stub Tenons.

When the mortise-and-tenon joint is combined with housing, it is called the housed tenon, as illustrated in Figs. 34*b* and *c*. The housing gives additional bearing surface to resist the lateral displacement of the timber having the tenon. A drip hole is provided to drain the mortise of water that enters the joint when exposed to the weather. The stub tenon is illustrated in Figs. 34*d* and *e*, the former being at the head of a pile. When the joint is at the ends of both timbers, an eccentric stub tenon is sometimes

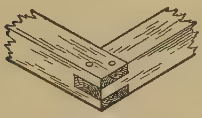


FIG. 34*f*.—Open Mortise.

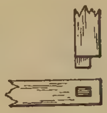


FIG. 34*g*.—Eccentric Stub Tenon.

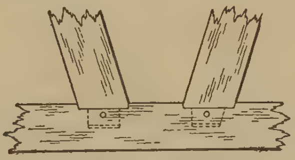


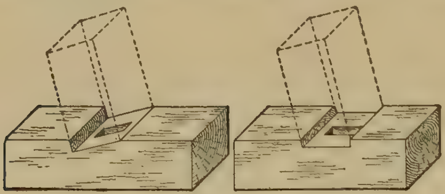
FIG. 34*h*.—Tenons on Batter Posts.

used as in Fig. 34*g*, whereas in other cases an open mortise is employed as in Fig. 34*f*. See also the corner of a car underframe in Fig. 19*g*. Figure 34*h* shows two forms of housed stub tenons when an inclined post meets a horizontal sill as in a wooden trestle bent. Figures 34*i* and *j* show two forms of housing for the housed tenon; in the first one the bearing surface for the shoulders is perpendicular to the pressure of the post, and in the second one the horizontal thrust of the post is resisted by the housing.

The dovetail tenon is illustrated in Fig. 34*k*. One side of the tenon is cut back into the form of a single dovetail, the mortise with receding sides is made wide enough to admit the broad end of the tenon, and then the tenon is held in place by a wedge driven along its back.

The tusk-and-tenon joint shown in Fig. 40*b* is practically obsolete. The tenon passes through the other beam and is held in place by a key. The tusk below the tenon is intended to form the principal bearing at the support. The double tenon is shown in the same illustration.

The double tenon as well as the tusk-and-tenon joints were formerly used in framing joists around a stairway, as in the joint between a tail beam and a header, or between a header and a trimmer. A tenon should not be used as a cantilever to carry a definite load. Its principal use is to prevent lateral displacement for a compression member in which an accidental blow may develop some flexure, or it may serve incidentally to resist tension while helping to maintain stiffness in a frame. Various forms of hangers have been devised in recent years to support one beam from another and thus to replace the joints mentioned (Art. 43).



Figs. 34*i* and *j*.—Sills of Trestles.

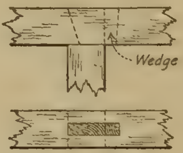


Fig. 34*k*.—Dovetail Tenon.

In discussing the different methods of framing the bents of wooden trestle bridges, a Committee of the American Railway Superintendents of Bridges and Buildings described the advantages and disadvantages of the mortise-and-tenon joint as follows: "The mortise-and-tenon joint seems to be the one in most general use, and if it were not for the extraordinary cost arising from the use of high-priced carpenter work, and the necessity for great accuracy in framing, it would be the best method for holding the timbers together. There are, however, several serious objections to this method, among which is the liability to rot on account of the mortise offering a receptacle for water, even though drained in a most approved manner. Every bridge man knows from his own experience and observation that the joints, especially where mortise and tenon are employed, decay long before signs of decay are visible at other points. This decay makes the renewal of the timber necessary, and as the balance of the stick is sound, much good timber is lost. This material can be, and generally is, used for shorter work, but at a cost nearly equal to new timber. Notwithstanding these defects in the mortise-and-tenon joint, it is the form generally in use and will probably continue to be as long as wooden bridge trestles are used. The advantages of its use are a maximum of strength and rigidity of the structure, which is the most important feature in the construction of a good trestle.

ART. 35. STEP JOINTS

Stepping consists of one or more notches with inclined sides or bearings arranged in the form of steps. Two forms of the single step are shown in Figs. 35*a* and *b* respectively, the bottom of the notch being horizontal in the former. This joint is used very extensively in all kinds of timber structures. See the upper ends of the braces in Fig. 72*b*, and the lower ends of two diagonal struts in Fig. 81*a*.

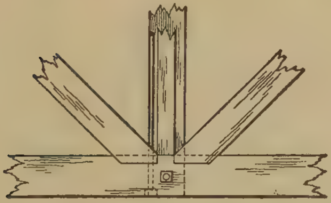


FIG. 35*a*.—Single Step Joint in Car Under-frame.

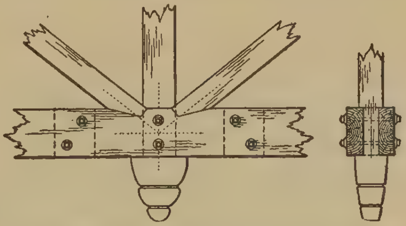
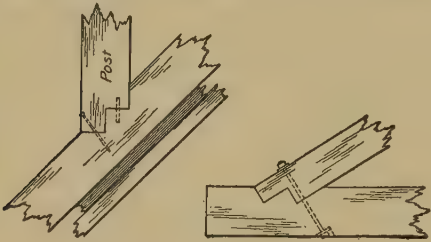


FIG. 35*b*.—Single Step Joint at Middle of Roof Truss.

Double steps are illustrated in a joint of an ore dock frame (Fig. 35*c*); in the end of a queen-post truss (Fig. 35*d*); and in a railroad bumping block (Fig. 35*e*). Another example is shown in the end joint of a roof truss in Fig. 35*f*. The same drawing from which this was copied also shows the use of three steps, although it is questionable whether more than two can be advantageously employed in such a joint. The objection to the preceding forms of double steps is the small shearing surface provided to resist the pressure on the second step. Figure 35*g* shows how this defect is overcome. In this case a much longer shearing surface is secured by making the depth *de* larger than *ac*, thus placing the shearing surfaces for the two steps at different elevations. In any case it requires very accurate workmanship to obtain the proper distribution of pressure on the two steps, but in Fig. 35*g* the eccentricity of the resultant pressure on the inclined compression member is materially less than in Fig. 35*f*.



Figs. 35*c* and *d*.—Double Steps.

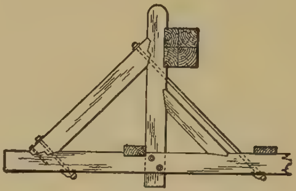
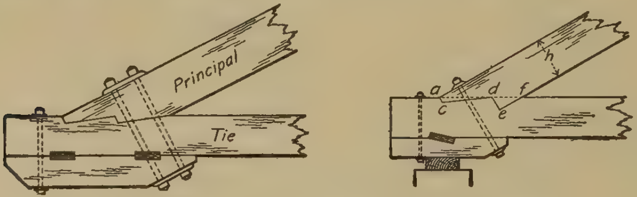


Fig. 35*e*.—Bumping Block.

The application of single, double, and triple steps in a single structure is made in a sectional drydock, illustrated in Engineering News, vol. 45, page 314 (inset), May 2, 1901.



Figs. 35*f* and *g*.—End Joints of Roof Trusses.

An example of multiple stepping may be seen at the end joints of the bowstring trusses of the Ferry Bridges of the New York Passenger Terminal, Central Railroad of New Jersey, Engineering Record, vol. 59, page 650, May 22, 1909. See also Fig. 79*a*.

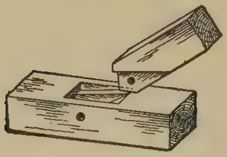


Fig. 35*h*.—Oblique Tenon.

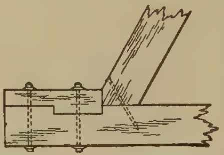
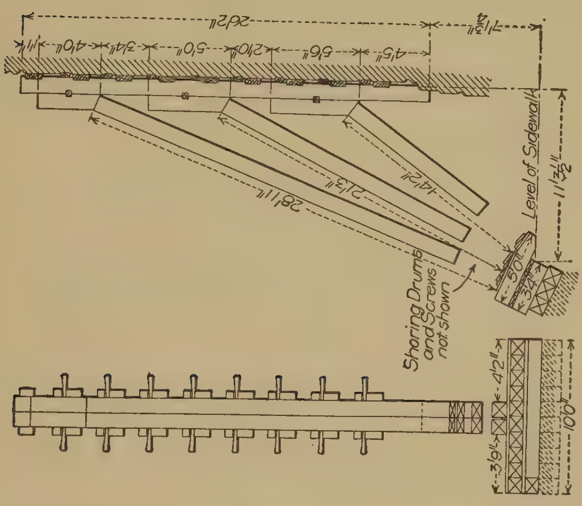
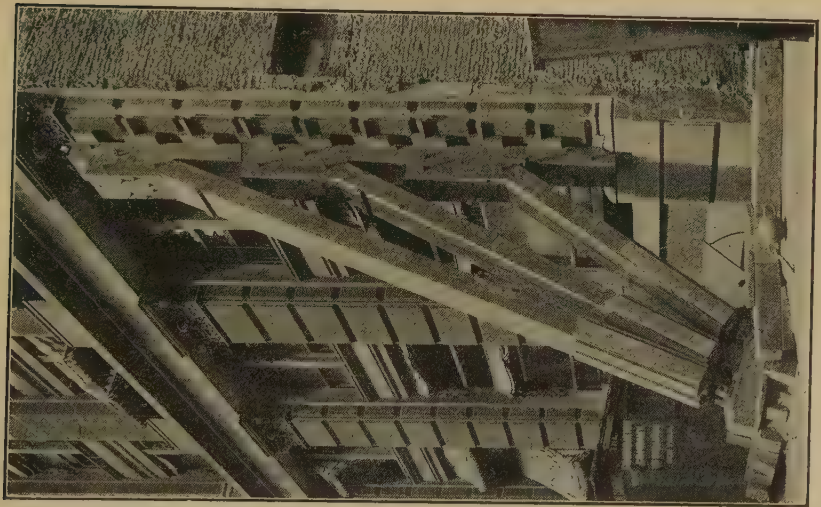


Fig. 35*i*.—Head Block.

When the mortise-and-tenon joint is combined with a step, as in Fig. 35*h*, it is called the oblique tenon. These joints are sometimes used in connecting struts to the chords of roof trusses but with the step omitted, so that the component of the stress must be taken by the end of the tenon. Whether this is allowable in good practice depends upon the magnitude of the stress in the brace and upon whether the tenon provides sufficient bearing area for the specified unit-stress.


26'2"
7'1¾"
4'1"
4'0"
3'4"
5'0"
2'10"
5'6"
4'5"
28'1"
21'3"
14'2"
Level of Sidewalk
11'3½"
5'0"
3'4"
Shoring Drums and Screws not shown
4'2"
3'9"
10'0"



FIGS. 35*j* and *k*.—Showing Ashlar Face Wall of the Farmer's Loan & Trust Company's Building in New York City.

Oak blocks were fitted into the rectangular arrises of the cut-stone blocks. Corresponding horizontal oak pieces were keyed into the pair of vertical timbers. Pairs of folding wedges were then placed between them, and the load transferred to the inclined pushers and jackscrews. See *Eng. Record*, vol. 58, page 480, Oct. 31, 1908.

In temporary construction, as for example in falsework, a head block is bolted to one timber to resist the longitudinal thrust of the other, thus replacing the steps. In Fig. 35*i* the head block has one table, for which keys are sometimes substituted. Usually, however, the head block is perfectly plain, and depends to some extent upon frictional resistance between it and the timber to which it is bolted. Another example of the use of head blocks with keys is illustrated in Figs. 35*j* and *k*.

Although it is not customary to consider the friction of the timbers in designing joints with steps it may be desirable to do so in special cases. The friction must be considered when plain head blocks are used, and it will resist a considerable part of the thrust on any block when the latter is not plain, provided the bolts are drawn up to their full working capacity.

Problem 35. Examine the framing of the Russell snowplow as illustrated in Railroad Gazette, vol. 23, page 743, and prepare a list of the different kinds of joints and fastenings used in its construction. Notice the timber knee and the manner in which it is employed.

Art. 36. Angle Blocks

On Plate III are given the detail drawings for four cast-iron angle blocks for a wooden roof truss, the use of which simplifies the framing of the timber braces. In every case each end of a brace is sawed off square as for a butt joint, and a hole is bored to engage the protecting pin on the angle block, thus holding the timber in place. The smallest one is a solid casting, whereas the rest are hollow with a uniform thickness of metal throughout.

Figure 36*a* shows another pattern of an angle block similar to the one mentioned in the preceding paragraph. In this case both ends are open. Holes are located in the middle of each side into which dowels may enter. The dowels are driven into the ends of the braces to hold them in place. The radial webs materially strengthen the block, which is intended for Howe trusses of spans less than 100 ft. For larger spans the angle blocks are more elaborate in form and contain tubes passing through the chord, so that the stresses in the vertical rods are transmitted directly through the angle block to the braces, thus avoiding the compres-

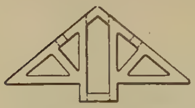


Fig. 36*a*.—Angle Block.

sion of the chord timbers on the side of their fibers. The details of separate rectangular tubes, as well as of two forms of angle blocks with open ends for a short span Howe truss bridge on an electric railway, are given on the inset accompanying an article in Engineering News, vol. 35, page 27, January 9, 1896.

The dimensioned detail drawings of a number of angle blocks and other castings for a temporary wooden roof truss used for exposition purposes may be found in Engineering News, vol. 28, page 605 (inset), December 29, 1892. Some of them are more complicated, since the type of truss adopted is not as well adapted to construction in timber as to construction in steel throughout. The same inset plate shows a cast-iron plate with short projecting pins or nipples on one side. A pair of such plates are used to connect a timber member to metal rods by means of a much smaller pin than can be employed if the bearing of the pin is directly on the wood. The projecting pins distribute the stress from the plate over a number of cylindrical bearing surfaces on each side of the timber. Angle blocks of wood are shown in illustrations contained in Arts. 72, 73 and 81.

PROBLEM 36. Consult Chapter IV in Keep's Cast Iron—a Record of Original Research (John Wiley & Sons) and ascertain the effect of different shapes in molds on the crystallization of cast iron. State the principles which must be observed in the design of castings to avoid planes or surfaces of weakness.

ART. 37. METAL SHOES

The details of an end joint of a king-post truss shown in Fig. 37*a* has one of the most effective forms of shoe now in use for the inclined upper chord when the stress is too large to be resisted by double steps of the form given in Fig. 35*g*. The shoe is forged from a steel plate to fit the end of the chord, the right end being bent down to form a lug which engages a notch in the lower chord.

In designing the shoe, provision must be made for sufficient bearing at the toe and at the lug, while the metal must have sufficient thickness to resist the bending moment due to the lug acting as a cantilever. The pressure at the toe and against the lug equals the horizontal component of the stress in the upper chord. The horizontal plate is subject to tension, but the unit-stress is very much less than that at its right end, which is due to flexure.

The shrinkage of the timber will release the tension put in the bolt when it is first drawn up, and after that the shoe has to move some distance horizontally before the bolt is again stressed in tension. Before that occurs, however, it will be subject to flexure on account of the pressure of the timber on the side of the bolt. These considerations indicate that the bolt and shoe cannot act together in resisting the horizontal thrust of the upper chord. The sole function of the bolt is therefore to keep the members in place. The hole should be a little larger than the bolt, and because the pressure of the upper end of the shoe increases the stiffness of the fibers of the timber, the bolt should be drawn up to its approximate safe tensile strength.

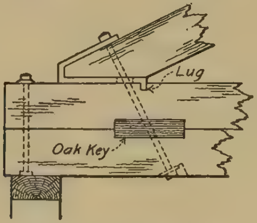


Fig. 37*a*.—Forged Steel Shoe.

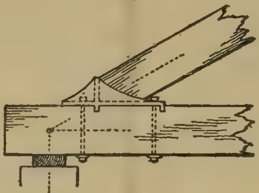


Fig. 37*b*.—Cast-iron Shoe.

A forged steel shoe can be designed to resist effectively the thrust at the end joint of the largest wooden roof truss that it is practicable to build. The plate may be extended, and one or more ribs riveted to it, as shown in Fig. 72*b*, or an extra bent plate may be added as on Plates I and II. A thinner plate may be used when the lug is omitted and only ribs are riveted to the plate. It is very essential that the lug or end rib be held down either by the pressure of the timber above it or by a hook bolt through the lower chord.

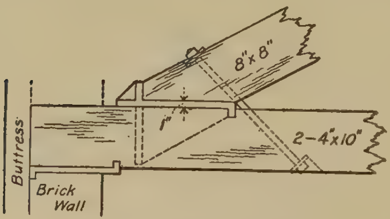


Fig. 37*c*.—Cast-iron Shoe.

Figure 37*b* shows a cast-iron shoe having two lugs to engage notches in the lower chord. The horizontal thrust is received by a transverse web which is connected to both side plates and to the horizontal one. The vertical pressure of the shoe on the lower chord is larger at the left end than at the right, since the center line of the upper chord intersects the horizontal resultant reactions against the lugs on the left of the middle. Another form of cast-iron shoe is given in Fig. 37*c*. As indicated on the diagram the lower chord consists of two timbers separated one inch which equals the thickness of the vertical triangular web of the shoe that lies between them and which engages the inner side of each timber by its projecting lugs. Another lug engages the top of each timber. The transverse web plate which receives the thrust of the upper chord is stiffened by a small triangular web.

ART. 38. PRESERVATION OF JOINTS

The joints of a wooden structure which is exposed to the weather are always the first to show incipient decay. The moisture enters between the surfaces of contact and remains after other parts of the timber become dry. It especially weakens the fibers at the ends of any timber by saturating the cell walls of the wood at the interior as well as at the surface of the stick. For this reason the diagonals of a Howe truss fail by crushing at their bearing on the angle blocks. Alternate wetting and drying cause rapid decay known as wet rot.

To increase the durability of a structure it is therefore important to treat the contact surfaces of timber at the joints with some preservative material. For this purpose creosote oil, common tar, and asphaltum, when applied very hot, are valuable and comparatively cheap. White lead and linseed oil are used to some extent, but are more expensive. Bolt holes or other holes bored in any part should be thoroughly saturated with the preservative, while all bolts, including drift bolts, should be warmed and coated with the same material. Coal tar may also be used for bolts and holes, no heating being necessary.

When the conditions are such as to make it economical to treat the timbers throughout, the framing should be done before subjecting the timbers to the preservative process. If any additional cutting is required after treatment, the exposed surface should be given several coats of the preservative with the brush, sufficient time being allowed for each coat to be absorbed by the wood before the next one is applied.

Various patented preparations, sold under the names of avenarius carbolineum, fernoline, woodiline, etc., are used to a considerable extent for the preservation of wood, all of them being either applied with the brush or by open tank treatment. To obtain the best results the materials are applied hot.

Wood preservation is made the subject of comprehensive study and investigation by one of the standing committees of the American Railway Engineering Association. See the first report in vol. 10 of the Proceedings (1909) giving statistics, considering preservatives, with specifications, adaptability of woods for treatment, the strength of treated timber, and treating processes.

A marked difference in the effect of oak and yellow pine wood upon iron bolts was discovered upon examining an old wooden bridge which was removed in 1902. Rods $1\frac{1}{4}$ in. in diameter were found to be reduced by corrosion to a diameter of about $\frac{1}{2}$ in. where they passed through oak blocks, whereas there was no perceptible corrosion where they passed through the yellow pine timbers.

CHAPTER III

WOODEN BEAMS AND COLUMNS

ART. 39. DESIGN OF WOODEN BEAMS

WOODEN beams are nearly always rectangular in section. When they are designed for strength only, the width and depth must be determined so that the unit stress in the outer fiber and the horizontal shear at the neutral surface shall not exceed their respective safe values. When only the stress in the outer fiber is considered, it is shown by mechanics that $bd^2 = 6M/s$; in which b and d are the width and depth of the beam respectively, M is the maximum bending moment, and s the unit stress in the outer fiber. Since the strength varies as the square of the depth d, the value of d should be assumed first, and then that of b found from the value of bd^2.

When only the unit shearing stress is considered, $bd = 3V/2s_h$, in which V is the maximum vertical shear and s_h the unit stress in horizontal shear. If the same security is desired in regard to both of these stresses, the value of d is found on dividing the former by the latter equation given above, whence $d = 4\ s_h/sV$. For a simple beam having a span l and a load W uniformly distributed, its value is found to be $d = ls_h/s$, or $d/l = s_h/s$. These equations show that the depth of the beam is directly proportional to the span, and that the ratio of depth to span is the same as the ratio of the allowable unit stresses at the neutral surface and in the outer fibers respectively. For structural timbers this ratio ranges from one-eighth to one-fourteenth. When, however, the entire load is concentrated at the middle, the ratio becomes $d/l = 2\ s_h/s$. Similar relations may be determined for any beam or condition of loading by using the values of the maximum bending moments and vertical shears, which may be found in the manufacturers' handbooks or in any textbook on mechanics.

For smaller depths than those given in the preceding paragraph, the strength of the beam is governed by the unit stress in the outer fiber, whereas for larger depths the strength depends upon the horizontal shear.

When the conditions under which beams are employed require them to be designed for stiffness, the dimensions of the cross-section must be found in such a way as to keep the deflection within a given ratio to the span. These conditions apply to joists or beams which support a plastered ceiling in a building, or a line of shafting in a machine shop or factory, or to stringers in wooden railroad bridges.

The deflection is usually limited from 1/360 to 1/480 times the span for ceilings, and to 1/200 of the span for the stringers in railroad bridges and trestles. In mill construction, where the deflection of floors must be limited on account of supporting lines of shafting, the allowable deflection, according to C. J. H. Woodbury, is that which causes the beam to bend into a curve with an average radius of 1250 ft.

In designing the beam by means of the formula for deflection, it is important to remember that the modulus of elasticity for a load applied continually is only about one-half as large as for a load which is removed within a few minutes after its application. The dead load is, therefore, to be doubled and added to the live load, the sum being substituted for the load W in the deflection formula, provided the live load is also distributed with at least approximate uniformity. Otherwise, it is necessary to solve the problem by trial. In this case the approximate size of the beam may be found by considering the live load only, afterwards computing the deflections for the dead and live loads separately by means of the respective formulas, and then comparing their sum with the allowable deflection. In some cases it should also be considered whether a beam is liable to receive its maximum load when still unseasoned, or only after it has been allowed to season.

Since the stiffness of a beam of rectangular section varies as the cube of the depth and directly as the width, the least material will be required when the section is just sufficient to keep the unit stress in horizontal shear within the safe limit. It is assumed that the unit stress in the outer fiber is usually also safe when this condition is fulfilled. When, however, the depth exceeds 12 in., the least material may not give the least cost, for the cost of dimension timber is a function of the depth or section area, as well as of the volume expressed in feet board measure.

If c_1 and c_2 denote the costs per foot board measure for two beams of depths d_1 and d_2 respectively, it may be readily shown that the total costs of the beams are to each other as c_1d_2 is to c_2d_1, provided the beams are of equal strength depending upon the stress in the outer fiber, the span and loading being the same. If c_3 and d_3 are the corresponding cost and depth of a third beam having the same deflection as the first one, their total costs are to each other as $c_1d_3^2$ is to $c_3d_1^2$. It will be found, accordingly, that for depths less than a certain value the beams must be designed by the formula for strength, and for greater depths by the deflection formula. As previously noted, the beams must be safe in all cases with respect to horizontal shear.

It is shown by mechanics that the elastic resilience of a beam is a measure of its resistance to external work or of its capacity to absorb shock. The resilience of a beam increases with its section area, and if only this function of the beam be considered, it is immaterial whether the long or the short side of the section is placed vertical. When two beams have similar cross-sections, their resiliences are directly proportional to their volumes.

The most resilient beam of rectangular cross-section which can be cut from a circular log has its width equal to its depth (see Fig. 39*a*). The strongest beam of rectangular cross-section which can be cut from a circular log has the relation between its width and depth as 1 to $\sqrt{2}$, or nearly as 5 to 7. If D is the diameter of the log, the sides of the cross-section are $b = D\sqrt{\frac{1}{3}}$ and $d = D\sqrt{\frac{2}{3}}$. These values indicate a simple graphical construction of the cross-section as shown in Fig. 39*b*, by dividing the diameter into three equal parts, erecting perpendiculars, and uniting the extremities of the perpendiculars and diameter. The stiffest beam of

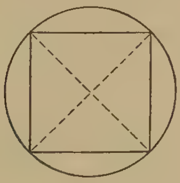


FIG. 39*a*.

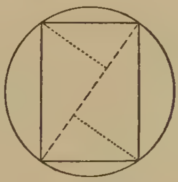


FIG. 39*b*.

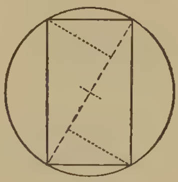


FIG. 39*c*.

rectangular cross-section which can be cut from a circular log has its width and depth related as 1 to $\sqrt{3}$, or as 1 to 1.732, or as 0.577 to 1. The width equals one-half of the diameter of the log and the graphical construction of the cross-section is indicated in Fig.

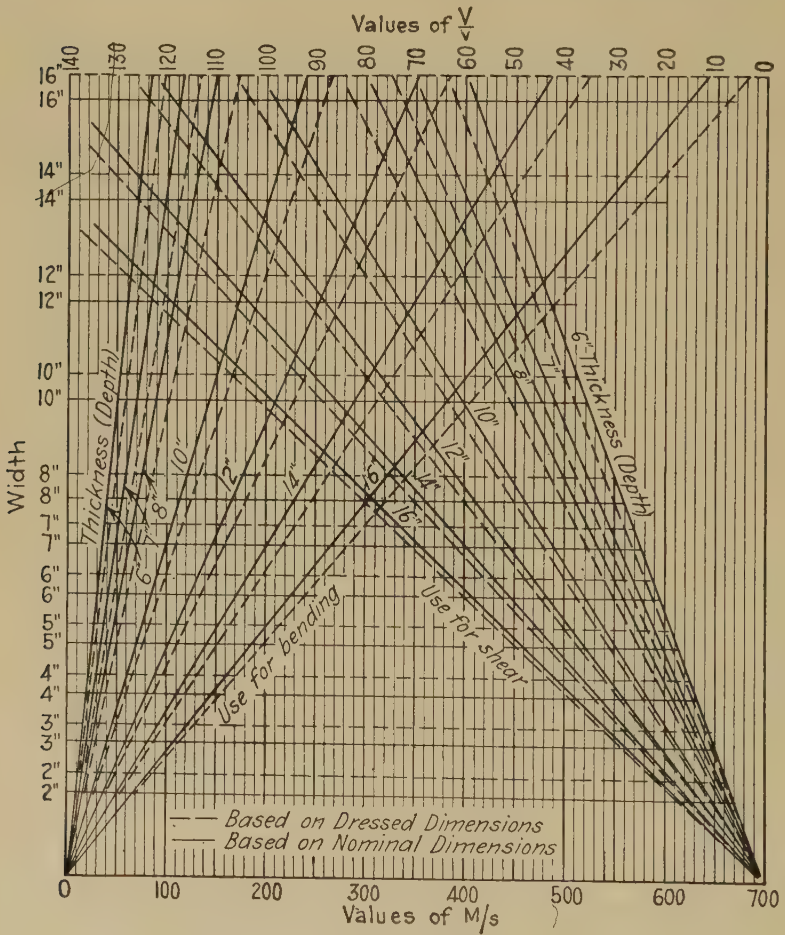


FIG. 39*d*.—Diagrams for Designing Beams.

39*c*. It is interesting to note that the depths in the three cases are related as $\sqrt{1} : \sqrt{2} : \sqrt{3}$, the width in each case being unity.

Where many beams have to be designed, it is convenient to use either tables or diagrams. In the Southern Pine Manual, published by the Southern Pine Association, tables have been prepared by the use of which beams may be quickly designed on the basis of bending, longitudinal shear and deflection. Data are given for a considerable range of unit stresses. Similar data for Pacific Coast woods are given in the Structural Timber Handbook on Pacific Coast Woods, published by the West Coast Lumbermen's Association.

By the use of Fig. 39*d* beams may be designed for bending or for shear, based on either dressed or nominal dimensions. For dimensions of 8 in. and over the dressed size is taken as $\frac{1}{2}$ in. short, whereas for smaller sizes the dressed dimensions are $\frac{3}{8}$ in. short. To design a beam for bending, calculate the value of the maximum moment, M, in inch-pounds and divide by the allowable unit fiber stress s in pounds per square inch. Enter the diagram with this value of $\frac{M}{s}$ and go vertically upward to the diagonal line giving the desired depth, d, and then horizontally to the left, where the required width, b will be found. The safe $\frac{M}{s}$ of any size of beam can be found by entering on the left at vertical distance giving the width, b, then moving horizontally to the right to the diagonal giving the depth, d, the value of $\frac{M}{s}$ being found by moving down vertically.

Likewise to get the size of beam necessary to carry a given shear, divide the total vertical shear, V, by the unit allowable shear, v. Enter the diagram at the top with this value of $\frac{V}{v}$ and go vertically down to meet any desired depth, d, and the required value of the breadth, b, will be found by moving horizontally.

ART. 40. FRAMING OF BEAMS

To ascertain the effects of bevels, notches, and mortises on the strength of beams, a series of tests of twelve forms of small wooden beams was made by William R. King. The span, form of beam, and manner of failure are shown in Fig. 40*a*. The beams were made of Tennessee poplar, and the timber was selected so as to secure a uniform quality. The breaking weight, as given, is the average of two tests which generally differ less than 5 percent.

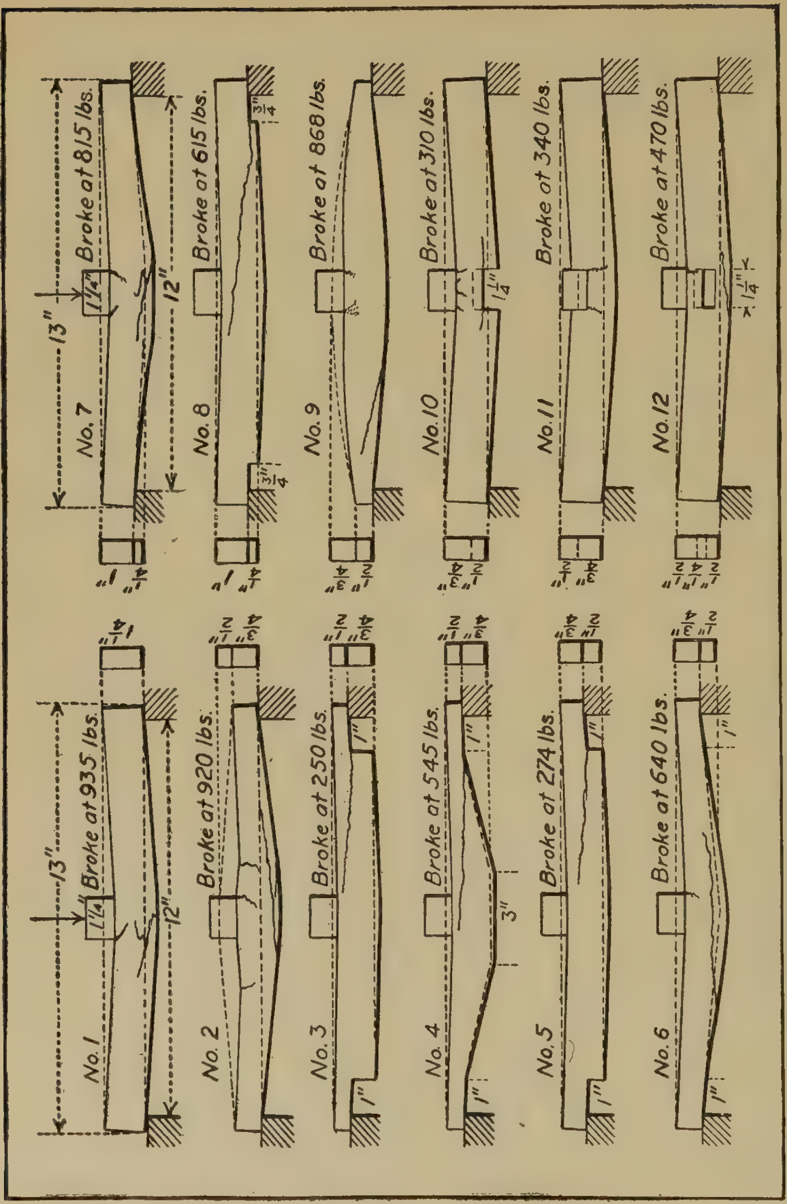

No. 1
Broke at 935 lbs.
13"
12"
1¼"
No. 2
Broke at 920 lbs.
No. 3
Broke at 250 lbs.
1"
No. 4
Broke at 545 lbs.
3"
1"
No. 5
Broke at 274 lbs.
1"
No. 6
Broke at 640 lbs.
1"
No. 7
Broke at 815 lbs.
13"
12"
1¼"
No. 8
Broke at 615 lbs.
¾"
No. 9
Broke at 868 lbs.
No. 10
Broke at 310 lbs.
1¼"
No. 11
Broke at 340 lbs.
No. 12
Broke at 470 lbs.
1¼"
1"
¼"
½"
¾"

FIG. 40a.—Tests Showing the Effect of Notches, Mortises, Etc., on the Strength of Beams.

One of the most remarkable facts shown is that a beam may be greatly strengthened by cutting away certain portions so as to conform more nearly to a beam of uniform strength. Thus, beams 3, 5, and 8 when cut away to the form of beams 4, 6 and 7 become respectively 118, 134, and 32 percent stronger. The weakness of the forms, with the shoulders near the supports, is mainly due to the great difference in stiffness on opposite sides of the shoulder and the resulting tendency to split, as indicated in the diagrams.

Charles Babcock made a similar set of tests at Cornell University with beams of the same clear span, but 1 in. by 2 in. in section, and cut down at the ends to a depth of 1 in. Five different kinds of wood were used: white pine, yellow pine, chestnut, ash, and oak; and the increase in strength due to cutting away the shoulders, so as to leave the full depth for a length of $4\frac{1}{4}$ in., ranged from 13 to 207 percent. The method of failure in each case was due to horizontal shear, or rather a combination of shear with tension across the grain.

On comparing the ultimate loads for beams 1, 3, and 8 in Fig. 40*a*, it is seen that a shoulder of only 20 percent of the depth reduces the strength 36 percent, while a shoulder of 60 percent of the depth reduces it 73 percent.

For Babcock's tests the percentage of reduction in strength due to a shoulder of one-half of the depth of the beam is as follows: white pine, 71; yellow pine, 66; chestnut, 70; ash, 70; and oak, 51. These facts emphasize the practical waste of material due to the common practice of cutting such shoulders on joists in buildings, and when conditions rarely justify it.

The notching of cross-ties over stringers when the spacing of the stringers materially exceeds that of the track rails, or of long cross-ties over deck trusses, leads to similar results, and this fact has been observed by bridge inspectors. The following extract is from an article by Justin Burns on the Inspection of Timber Structures: "If the Howe truss is a deck bridge, the track will probably be carried on ties resting on the top chords of the trusses. These ties are usually 14 or 16 in. in depth, their size, of course, being dependent upon the distance between trusses. They are notched down a few inches over the chord to prevent their lateral movement and these notches should be observed for cracks, or the splitting away of the lower portion of the tie.

Theoretically, the notched tie is as strong a beam as one of the same depth but unnotched, yet this square notch has a weakening effect, as the timber tends to split off below the notch."

Exception may be taken to this reference to theory, since theoretically as well as practically any radical change in cross-section in a member, whether it is subject to tension or to flexure, deflects the lines of stress from their normal direction and thereby weakens it. The loss of strength is due not only to cutting away material, but also to the development of secondary stresses.

The breaking loads for beams 10, 11, and 12 show the weakening effect of a notch or mortise at the middle of a beam, but the beam is not so reduced in strength as when a shoulder is made at the ends by cutting down the depth.

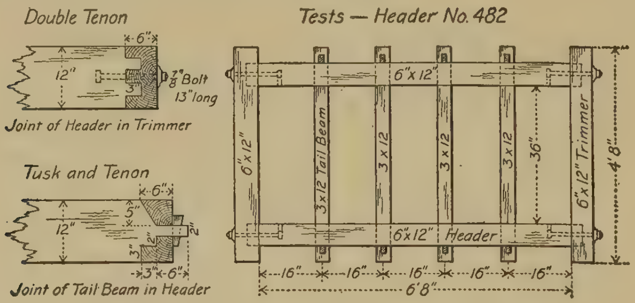

Double Tenon
6"
12"
3"
7/8" Bolt
13" long
Joint of Header in Trimmer
Tusk and Tenon
6"
5"
12"
2"
3"
3"
6"
Joint of Tail Beam in Header
Tests — Header No. 482
6"x12"
6"x12"
3x12 Tail Beam
3x12
3x12
3x12
36"
6"x12" Trimmer
4'8"
6"x12" Header
16"
16"
16"
16"
16"
6'8"

FIG. 40*b*.—Arrangement of Beams for Tests.

At a stairway or other opening through a floor, the beams on the sides of the opening are called trimmers; the cross beam which is near the head of a person passing up or down the stairway is called the header, and the joists or beams supported by the header are known as tail beams.

A series of tests was made at Massachusetts Institute of Technology to find the strength of timber headers loaded through tail beams framed into the headers by tusk-and-tenon joints. As the test was to be of the headers and not of the tail beams, the latter were made quite short. In some cases each end of the header was framed to the trimmer by a double tenon joint and held in place by a joint bolt, whereas in others the header was supported by a stirrup and held in place by a joint bolt. The dimensions of the joints and the general arrangement are shown in Fig. 40*b*, the number of tail beams ranging from 4 to 7.

All the headers gave way by splitting along the line of mortises cut to receive the tusks and tenons of the tail beams. In every case where the headers were framed into the trimmers by double tenon joints the end below the lower tenon also split off.

The general results show a reduction of strength in the hard pine headers varying from 50 to nearly 90 percent, due to the cutting involved in the method of framing. With a material where the shearing strength along the grain is relatively so small as in timber, any cutting whatever injures the strength and stiffness of the beam. It is therefore much better to avoid framing whenever possible and to use stirrups or some approved form of hangers, both for the tail beams and headers. The detailed record of the tests and half-tone illustrations showing the methods of failure were first published in Technology Quarterly, vol. 10, and afterwards reprinted in pamphlet form.

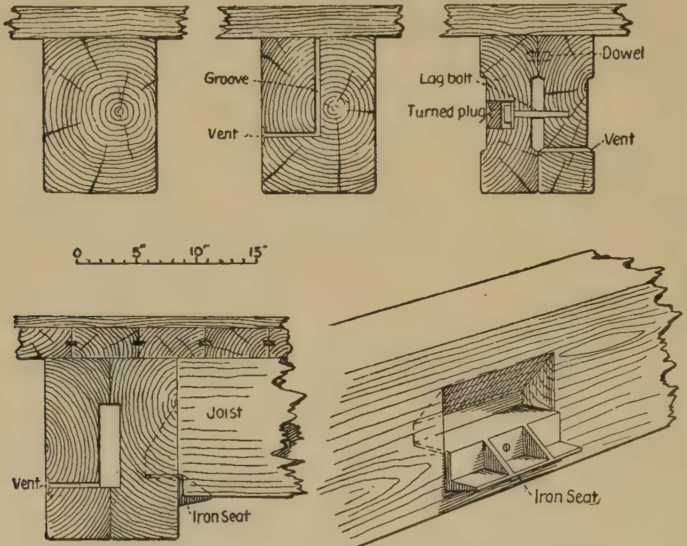
PROBLEM 40. Compute the ultimate value of the maximum horizontal shear in beam No. 1 in Fig. 40*a* and its probable corresponding value in beam No. 3.

ART. 41. CONSTRUCTION OF WOODEN BEAMS

Figure 41*a* shows the section of a large beam in which season checks extend outward from the heart since the heart dries out more slowly than the outer wood of more recent growth. Figure 41*b* shows a method of admitting air to the interior by means of a saw cut extending through a part of the depth and providing vent holes at intervals. Another method which is more frequently adopted is to cut the stick vertically into halves and to place the heart sides outward as well as to reverse one piece. This arrangement affords the additional advantage of inspecting the heart, which in mature trees begins first to decay.

Figure 41*c* shows the two halves with panels planed on the sides, thereby admitting air also to the interior. The vent holes are placed at the bottom of the central groove and can therefore drain off any water that may collect. They are joined by lag screws, which do not become loosened so much by the shrinkage of the wood as if bolts are used extending clear through. The dowels in the top help to equalize the deflection, and therefore also the loading in cases like that shown in Fig. 41*d*. The panels on the outside materially improve the general appearance and reduce the strength but slightly.

Although the methods described in the preceding two paragraphs permit better inspection and prevent serious checking in beams as the timbers dry out in the structure, there is an additional advantage in building up a beam or girder of two or more sticks placed side by side, since the smaller sticks cost less than the larger ones, and are more readily obtainable in the market. Such a beam is sometimes called a compound or composite beam, and the extreme case is one in which a series of planks are fastened together side by side with spikes or bolts. If every plank extends the full length of span, the combination is fully equal to and in most cases stronger than a solid beam of the same external dimensions, since with proper inspection large knots and other defects in the inner planks, which materially impair the strength, will cause their rejection, whereas in a solid stick only the outer surfaces can be thoroughly inspected. It is necessary, however, in case spikes are employed to fasten the planks together, that some through bolts must be provided to prevent the outer planks from buckling.



FIGS. 41*a*, *b*, *c*, *d*, and *e*.—Sections of Large Wooden Beams.

Figure 41*f* shows the general plan of one span of a pile trestle with ballasted floor. The floor consists of creosoted timber and each stringer is made up of two pieces, 3 by 16 in., to secure more thorough creosoting than can be done with a solid stick. The

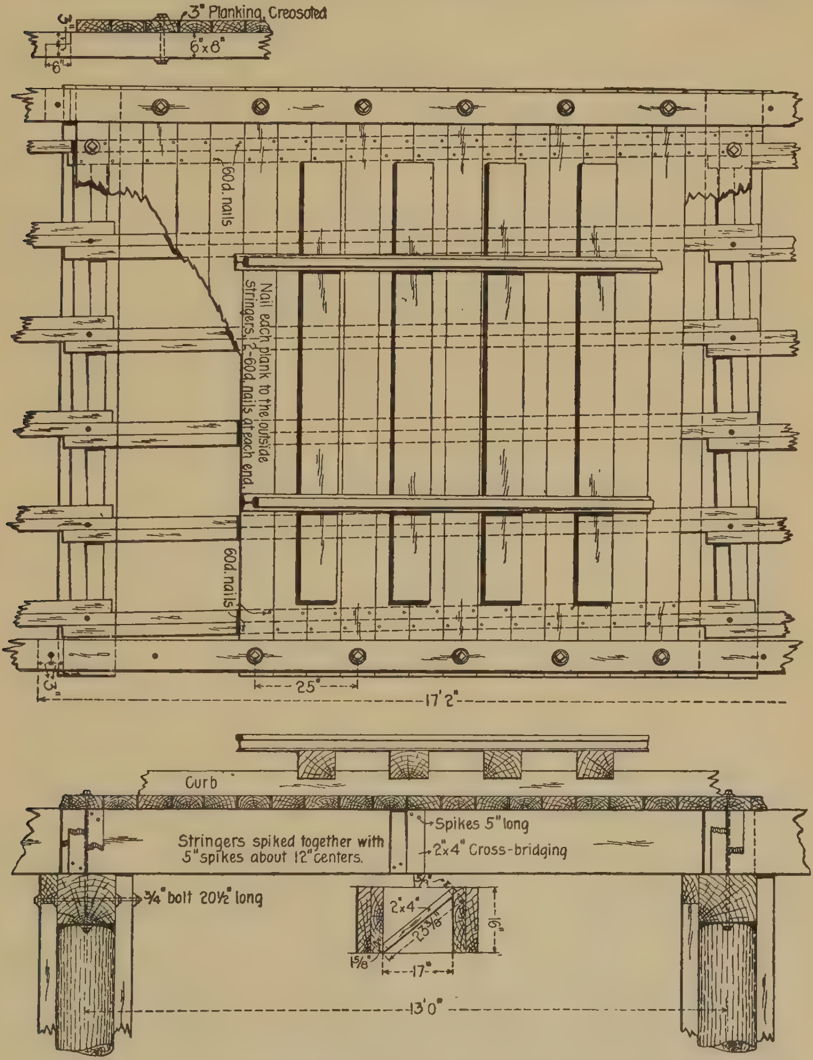

3" Planking, Creosoted
3"
6"x8"
6"
60d. nails
Nail each plank to the outside stringers, 2-60d. nails at each end.
60d. nails
3"
25"
17'2"
Curb
Stringers spiked together with 5" spikes about 12" centers.
Spikes 5" long
2"x4" Cross-bridging
3/4" bolt 20½" long
5/4"
2"x4"
23⅛"
16"
1⅝"
17"
13'0"

FIG. 41*f*.—Plan and Longitudinal Section of One Span of Pile Trestle with Ballast Floor.

sticks are spiked together with 5-in. spikes spaced about 12 in. apart; each stick extends over two spans, and is bolted at its center to the cap of the bent. The arrangement of the stringers shown avoids the necessity of cutting off the end of any stick which may be too long, since it is very objectionable to do any cutting or framing of timbers after they have been creosoted. It also has the advantage of allowing the floor to be readily strengthened by putting in additional stringers without disturbing the roadbed or existing stringers.

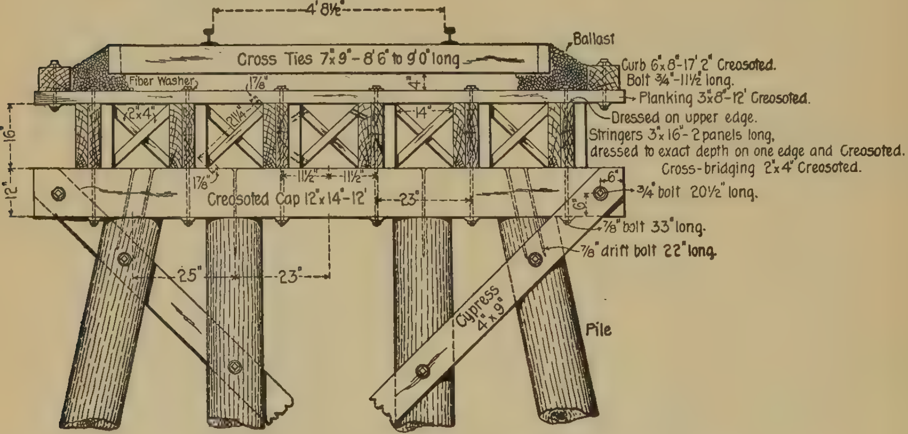

4′ 8½″
Ballast
Cross Ties 7″x 9″ – 8′ 6″ to 9′ 0″ long
Curb 6″x 8″–17′ 2″ Creosoted.
Bolt ¾″–11½″ long.
Fiber Washer
1⅞″
4″
Planking 3″x 8″–12′ Creosoted.
Dressed on upper edge.
Stringers 3″x 16″– 2 panels long,
dressed to exact depth on one edge and Creosoted.
Cross-bridging 2″x 4″ Creosoted.
2″x 4″
21¼″
14″
16″
12″
1⅞″
11½″
11½″
Creosoted Cap 12″x 14″–12′
23″
6″
6″
¾″ bolt 20½″ long.
⅞″ bolt 33″ long.
⅞″ drift bolt 22″ long.
25″
23″
Cypress 4″x 9″
Pile

FIG. 41*g*.—Cross-section of Pile Trestle with Ballast Floor.

The advantages of using two timbers instead of one in trestle construction are indicated in the report of a Committee of the American Railway Superintendents of Bridges and Buildings on Frame and Pile Trestles: "One method of obviating many of the defects in the mortise-and-tenon joints is by the use of double caps, posts, and sills. That is, in place of using 12- by 12-in. timbers, two timbers 6 by 12 in. can be used; and, being properly fitted and securely bolted together, they not only give good results as to strength and durability, but expose all defects frequently found in the center of large timbers. Also, being of only one-half the size and weight, they can be handled much more rapidly and consequently much more cheaply. The parts to be renewed are rendered more accessible, this being especially the case where trains are to be carried while the renewals are made. No other method of constructing trestles secures such great economy in renewals, and it has all the advantages of the ordinary mortise-and-tenon joint." One distinctive feature of this construction requires especial emphasis, namely, that any one piece may be renewed without disturbing the rest of the structure. For a discussion on the strength of posts composed of more than one piece, see Art. 53.

On account of the fire risk involved with narrow open spaces between timbers in buildings it is considered better to bolt them together as tightly as possible, depending upon shrinkage and barometric changes to give all the necessary opportunity for access of air to the inner surfaces. See letter of John R. Freeman in Engineering Record, vol. 55, page 194, February 23, 1907, and page 330, March 9, 1907.

Problem 41. Compute the modulus of rupture, ultimate horizontal shear, and modulus of elasticity of the beam composed of three oak liners bolted together, the data for which are given at the bottom of the table in Railroad Age Gazette, vol. 46, page 1489, June 24, 1909. Compare these results with the corresponding unit-stresses given in the table in Art. 89.

Art. 42. Packed Stringers

A packed stringer is made up of a number of pieces of equal depth placed side by side but not in contact, and bolted together. Such stringers are commonly used in the ordinary type of wooden trestles with open floors. A stringer is placed directly beneath each rail. They vary in depth from 12 to 20 in.; a depth of 16 in. is most frequently used, and the modern heavy loading requires depths of 17 to 20 in. For light loads two sticks may be used under each rail, but for the heavier loads three or four sticks are necessary. The span ranges from 12 to 16 ft.

As trestles are exposed to the weather, the sticks cannot be placed in contact, since moisture would collect between them and start decay. To avoid this they are spaced from 1 to 2 in. apart, although sometimes the clear distance is even greater. The spacing is secured by cast-iron separators or spools, or by wooden packing blocks. The iron devices are preferable as they are not so liable to gather moisture. Numerous forms of separators and spools are employed, Figs. 42*a*, *b*, and *c*, showing one form of spools and two forms of separators. A substitute for a separator consists of two washers with their larger flat sides against the timbers and the smaller ones in contact. Figure 77*b* shows the plan of a steel packing plate which also serves as a splice plate to aid in holding together the ends of the stringers. The plate is separated from the timbers by thin cast-iron separators. A packing splice is also indicated in Fig. 42*d*. When no bolsters are used, it extends below the stringers and is notched over the cap to hold the stringer in place. Four $\frac{3}{4}$-in. bolts are usually employed at each joint. Timber packing blocks are objectionable on account of holding moisture and promoting wet rot in the adjacent sticks.

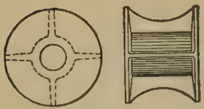
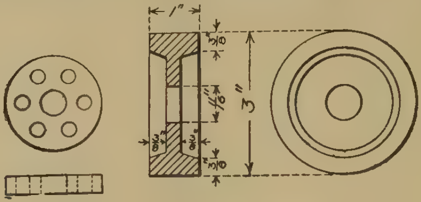


Fig. 42*a*.—Packing Spool.



Figs. 42*b* and *c*.—Separators.

When it is possible to secure the timber of the requisite length, the sticks composing the stringers should extend over two spans and break joints over the caps of the bents. The arrangement of the bolts shown in Fig. 42*d* is such that when used with steel splice plates the stringers have been known to carry a train in safety when one supporting bent was washed out. The lower part of the splice plate is in tension in such an emergency, and hence the lower bolts are placed farther apart to give a larger resistance to shearing or splitting in the ends of the timbers.

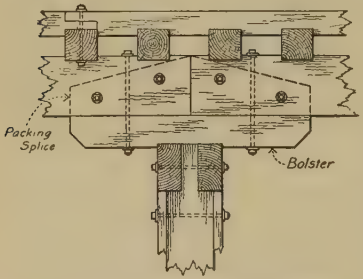


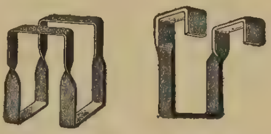
Fig. 42*d*.—Packing of Stringers.

To secure the stringers against shifting laterally or longitudinally the best practice is to fasten the middle of each continuous piece to the cap by a drift bolt. The bolts should extend into the cap about two-thirds of its depth.

Problem 42. Consult Foster's Wooden Trestle Bridges and prepare a table giving the spacing for the sticks composing the stringers as shown on the various standard plans.

ART. 43. BEAM HANGERS

The first metallic hanger to be introduced as a substitute for the joints, referred to in Art. 34, and still in use, is the stirrup or stirrup iron. The single form illustrated supports one end of a joist; the double form supports two joists or beams on opposite sides of a beam or girder (see Figs. 43*a* and *b*). In determining the section of steel flat bars to be used, the most important considerations are to provide sufficient bearing area on the side of the fibers for the joist or beam, and to make the bars of sufficient thickness to resist the bending moments at the center or edges of the lower bearing and at the edges of the upper bearings. If the bars are too thin, they will bend materially, and thus fail to distribute the bearing pressure uniformly.



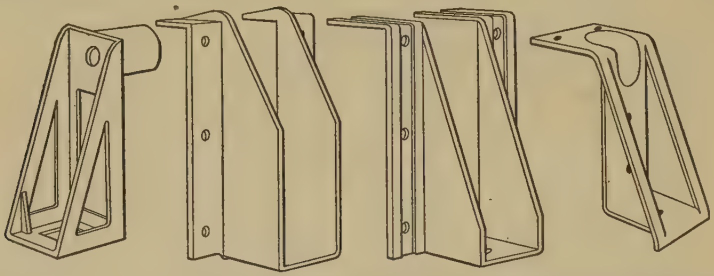
FIGS. 43*a* and *b*.—Double and Single Stirrup.

For example, let it be required to design a stirrup to support a load of 2100 lb. from a girder 6 in. wide. The spruce beam which bears on the stirrup is 4 in. wide. For an allowable compression on the side of the fiber of 250 lb. per sq. in. the width of the stirrup bar must be $2100/(4 \times 250) = 2.10$ or $2\frac{1}{4}$ in. The pressure of the stirrup on top of the girder may be assumed to vary from zero at the rear edge to its maximum value at the front edge. The load on each arm of the stirrup is $2100/2 = 1050$ lb., and the lever arm of the reaction of the girder against the stirrup arm is $6/3 = 2$ in., making the bending moment $1050 \times 2 = 2100$ in.-lb. The resisting moment of the stirrup section at an allowable unit stress of 16,000 lb. per sq. in. is $16{,}000 \times 2.25t^2/6 = 6000t^2$ in.-lb., t being the thickness of the bar in inches. Equating the two moments, t is found to be 0.592 or $\frac{5}{8}$ in.

The average compression per square inch on the side of the fibers of the girder is $1050/(2.25 \times 6) = 77.7$ lb. and the maximum compression 155.4 lb., which is below the specified limit. If it were assumed that the pressure at the front edge of the girder is 250 lb. per sq. in., then the point of zero pressure would be less than 4 in. from that edge, and the bending moment thus produced would be less than that computed in the preceding paragraph. If the load on the stirrup bar under the end of the beam is regarded as uniformly distributed, the bending moment in the bar at the center of the beam is also found to be less than that previously computed.

Stirrups are frequently used to support the purlins of a roof truss when conditions do not favor resting the purlins directly on top of the upper chord, as well as to support the joists of a ceiling from the lower chord of a truss. The use of a stirrup in suspending a beam below a window cap in a factory building is illustrated in Engineering Record, vol. 43, page 526, June 1, 1901. The same illustration shows a double stirrup, with lag screws inserted below to keep the beams from sliding off their seats.

A number of patented forms of beam hangers of wrought iron, malleable iron, or steel have been introduced, and are used in structures of the best design. Figure 44*d* shows the Goetz joist hanger, which is made of wrought iron. The illustration shows the hanger resting on cast-iron anchor plates, and when so arranged is called a wall hanger. The ends of the joist hanger are inserted into holes bored into the supporting beam or girder above the neutral axis, and as the ends are bent down the hanger tends to keep in position.



FIGS. 43*c*, *d*, *e*, and *f*.—Duplex, National, Van Dorn, and Ideal Beam Hangers.

Figure 43*c* shows the duplex hanger for a narrow beam, which is made of malleable iron; another form is for a joist meeting the supporting beam at an angle of 45 deg.; a third form is adapted to large size timbers, being used in pairs. The cylindrical bearing of this hanger has a horizontal axis. The projecting lug at each side is intended to engage a notch in the joist. In computing the end bearing of the joist it will be noted that an allowance must be made for the opening in the bottom of the hanger. This type of hanger was first placed upon the market in 1895. In computing the safe load for the Goetz or the duplex hanger it may be assumed, according to the results of tests, that it is limited only by the safe bearing value of the cylindrical bearing surfaces on the side of the fibers of the supporting beam. As shown in Art. 24, the effective bearing area equals the horizontal projection of the cylindrical surface when the direction of pressure is perpendicular to the fibers.

Figures 43*d*, *e*, and *f* show three forms of pressed-steel joist hangers. They are known as the national, Van Dorn, and ideal steel joist hangers, respectively. In each case the hanger is supported on top of the beam, but in Fig. 43*d* the resistance to bending at the upper corners is increased by extending the vertical plates above the adjacent bearing plates. The increased stiffness thus secured helps to distribute more uniformly the pressure on the top of the beam. In Fig. 43*f* the increased resistance to bending is obtained by sacrificing the bearing surface. Figure 43*g* shows an additional form of hanger which resembles the stirrup in some respects. It aims to distribute the pressure in the supporting beam over a larger surface. The elements of weakness in hangers when not designed for the loads they have to carry have been demonstrated by building failures. See Engineering News, vol. 48, page 420, November 20, 1902; and also vol. 49, pages 58 and 128, January 15 and February 5, 1903.

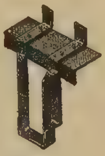


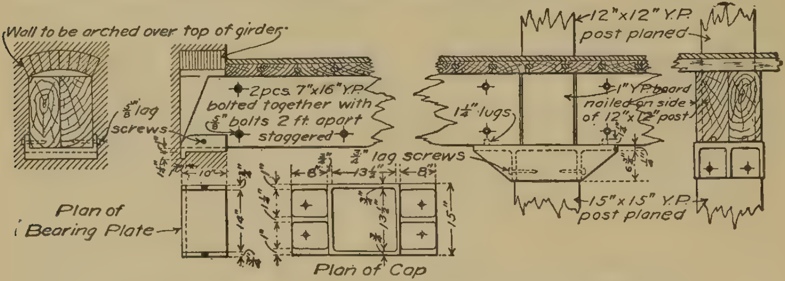
FIG. 43*g*.

PROBLEM 43. Secure the catalogues issued by manufacturers of beam hangers and make a critical comparison of the results of tests given, and note the manner in which the load is applied in case a hanger is not tested in a regular testing machine.

ART. 44. ANCHORAGE OF BEAMS TO WALLS

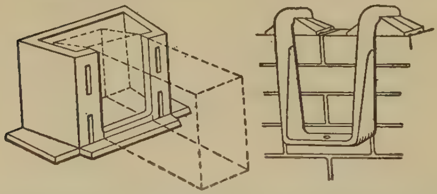
A large number of devices have been used in practice to anchor a beam or joist to a wall which supports its end. Some of them are of such flimsy construction that they can be regarded as anchors in appearance only. Some of those which are effective in certain respects are dangerous in others, since they overturn the wall in case of fire if the beam gives way. In the best construction the anchorage is arranged to release the beam in such an emergency without damage to the wall.

In Fig. 37*c* the beam is supported on a cast-iron wall plate which has a lug at the rear to hold the plate in position, and another lug inside of the wall to engage a notch in the bottom of the beam. Since the beam has no bearing on the right of the notch it is weakened by it (Art. 40). This defect may be remedied by locating the lug nearer to the end of the beam as in Fig. 50*e*. Sometimes it is placed at the middle of the plate, but in this case the longitudinal shearing surface behind the lug is too small to insure an effective anchorage for the beam. In Fig. 44*a* the beam is held longitudinally by lag screws on the sides, but with this detail the beam is only released by splitting or shearing its end. Instead of a lug to hold the plate in position, the plate is often made trapezoidal in form to fit a corresponding recess in the wall. In both of the preceding figures the wall is arched over the beam in order to secure ventilation around it. Cast-iron plates or flat stones are often substituted for the arch.


Wall to be arched over top of girder
5/8" lag screws
2 pcs. 7"x16" Y.P. bolted together with 5/8" bolts 2 ft. apart staggered
10"
14"
Plan of Bearing Plate
3/4" lag screws
8"
13 1/2"
8"
15"
Plan of Cap
12"x12" Y.P. post planed
1 1/4" lugs
1" Y.P. board nailed on side of 12"x12" post
15"x15" Y.P. post planed

Figs. 44*a* and *b*.—Cast-iron Bearing Plate and Post Cap.

The Goetz-Mitchell system of anchoring a beam is illustrated in Figs. 44*c* and *d*. The anchor consists of a cast-iron box of dovetail shape to lock into the wall, and with a lug on the bottom plate over which the joist or beam is notched. For the wider boxes interior side guides are provided which permit a free circulation of air to prevent dry rot. The end of the beam is cut at an inclination to enable the falling beam to release itself from the anchorage. The box is provided with a cover to support the masonry directly above it. The box anchor prevents fire or



Figs. 44*c* and *d*.—Goetz Box Anchor and Wall Hanger.

smoke from passing through the wall, and also protects the beam from the danger of fire on the opposite side of a party wall or from adjacent flues. The joists upon one side of a party wall can all burn out while those upon the opposite side hold the wall in position. For heavy timbers the bottom of the box is extended to increase its bearing area on the masonry. The projection of the base plate beyond the lug gives additional bearing for the beam and thereby resists the tendency of the lower fibers to split off.

The introduction of this system of anchoring beams was recognized as a noteworthy improvement in building construction and an aid in the reduction of fire risk, by the award of a medal and premium by Franklin Institute in 1891. The value of an effective method of anchorage is also shown to be practical by the fact that insurance companies allow reduced rates of premiums.

Wall hangers are also frequently used for the support of beams. As shown in Fig. 44*d* the Goetz wall hanger consists of the beam hanger bearing on anchor plates of cast iron provided with anchoring lugs, made to set level on the wall and to fit the angle of the hanger arms or hooks. The duplex wall hanger differs from the beam hanger in replacing the hollow cylindrical bearing with a projection extending into the wall resembling an I-beam but with a rectangular opening cut out of its web and with its lower flange extended on each side to provide the necessary bearing on the masonry. The Van Dorn or national wall hanger is made by riveting a beam hanger to a steel plate which is as wide as a brick horizontally, and is then bent up for bonding into the wall. The former also has the plate bent down a short distance on the face of the wall (Fig. 44*e*).

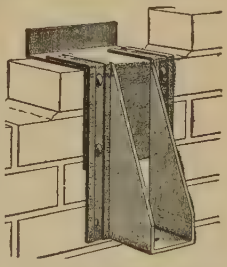


FIG. 44*e*.—Van Dorn Wall Hanger.

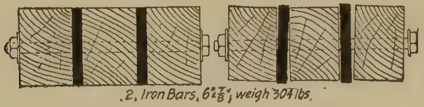
The use of wall hangers makes the load on the wall more eccentric and the anchorage is less effective than with box anchors. If the beams are fastened too securely to the hangers, the wall will be pulled over by the falling beams in case of fire.

For further illustrations see Art. 75.

PROBLEM 44. Find the compression per square inch on the bearing plates at both ends of the beam in Fig. 44*a*. To find the uniformly distributed load, take the allowable stresses in the outer fiber as 1950 lb. per sq. in., and the unit horizontal shear as 180 lb. per sq. in. The distance from the inside of the wall to the center of the column is 18 ft.

ART. 45. COMBINATION BEAMS

A combination beam is one in which two different materials, as, for example, steel and wood, are combined in its construction. The oldest form of strengthening a wooden beam without changing its form and size, or but slightly increasing the latter, consisted in inserting an iron plate between two sticks of timber or two plates between three sticks, the timbers and iron plates being thoroughly bolted together so as to act as a single member (Fig. 45*a*). This form is generally known as a flitch beam. A rectangular cross-section is not an advantageous form for metal to resist flexure, but when the flitch beam was introduced, the modern structural metal shapes were not in existence.



FIGS. 45*a* and *b*.—Flitch-plate Bolsters.

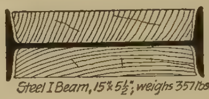


FIG. 45*c*.—Bolster.

The investigation and design of a flitch beam depends upon the equal deflection of its parts. It is shown in mechanics that for any given span and loading the deflection varies as W/EI in which W is total load, E the modulus of elasticity of the material, and I the moment of inertia of the cross-section of the beam. Since the wooden and steel parts have the same deflection, $W_w/E_wI_w = W_s/E_sI_s$, in which the subscripts w and s refer to the wood and steel respectively. For reactangular cross-sections of equal depth, the equation becomes $W_w/E_wb_w = W_s/E_sb_s$. Under the same conditions the relation between the unit stresses in the outer fiber is

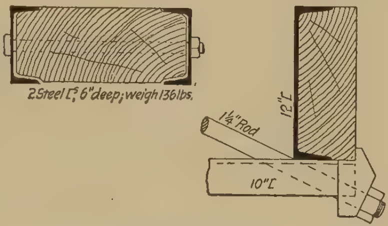
$$\frac{s_w}{s_s} = \frac{W_w}{b_w}\Big/\frac{W_s}{b_s}.$$

Combining these equations, the following relation is obtained,

$$s_w/s_s = E_w/E_s.$$

For example, if longleaf yellow pine and steel are combined in such a beam, with values of the modulus of elasticity of 1,600,000 and 28,000,000 lb. per sq. in., respectively, the timber can be stressed only to 914 lb. per sq. in., provided the steel is stressed to its allowable value of 16,000 lb. per sq. in. If the steel plate is omitted, the timber can be safely stressed to 1950 lb. per sq. in., in the outer fiber, when protected from the weather. This result indicates that under these conditions the efficiency of timber in a flitch beam is only about 47 percent; and since a steel plate has only about 70 percent of the strength of an I-beam having the same depth and sectional area, it is not economical to use a flitched beam when I-beams are available.

Figures 45*a* and *b* show the sections of a flitch-plate bolster; Figs. 45*c* and *d* of combination bolsters; and Fig. 45*e* of a side sill in car construction. They represent results of the earlier attempts to strengthen members of the underframe, in order to increase the carrying capacity of modern freight cars. Figures 45*b*, *c*, and *d* illustrate the effect of shrinkage of the timber. The longitudinal expansion and contraction of the metal, due to changes of temperature, cause additional stresses in their connections as well as in the component parts of the beam. When flitch beams are used in a building, in case of fire the metal plate will twist, and practically destroy the beam.



FIGS. 45*d* and *e*.—Combination Bolster and Side Sill.

PROBLEM 45. Consult Merriman's Mechanics of Materials, Art. 112, and study the order of design for a flitch beam.

ART. 46. DEEPENED BEAMS

When the load and span becomes too large for a single stick of an available commercial size, two general methods may be employed to build up beams of the requisite strength and stiffness. First, by placing side by side two or more sticks of the same depth, but relatively narrow, and bolting them together either with or without intervening spaces; and, second, by placing two or more sticks of the same width on top of each other and bolting them together with keys or brace blocks, as indicated in Fig. 46*b*, to prevent sliding of the contiguous surfaces.

The first type is described in Arts. 41 and 42. The second type is known by various names, including "built-up beams," "compound beams," or "keyed beams," as well as that adopted for the title of this article. Only two sticks are generally employed in deepened beams; although three sticks have been used occasionally, it is more economical in that case to substitute trusses.

If two timbers of the same width b, and the same depth d, are placed on top of each other, as in Fig. 46*a*, and the friction neglected in the surface of contact, the resisting moment of the timbers is $sbd^2/3$, in which s is the unit stress in the outer fiber. If the timbers are so connected by keys or brace blocks, as in Fig. 46*b*, that they act like a single stick of depth $2d$, the resisting moment becomes $2sbd^2/3$ thus doubling the combined strength of the separate timbers. The keys must effectively resist any tendency to sliding between the adjacent surfaces, so as to subject all the fibers of the upper timber to compression and those of the lower one to tension. Figure 46*c* shows to a large scale the end of a deepened beam, in which the brace blocks are narrower than the beam, the plan and sections of a block being given in Fig. 46*d*. The position of the end brace block with respect to the center of the support should be noted.

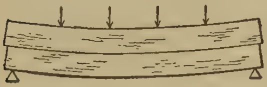


FIG. 46*a*.—Superimposed Beams.

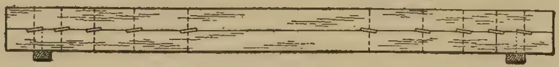


FIG. 46*b*.—Deepened Beam.

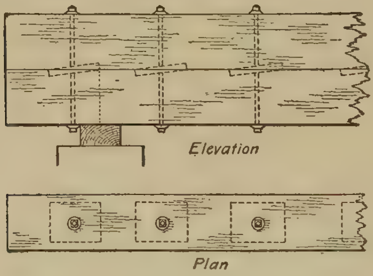


FIG. 46*c*.—Part of Deepened Beam.

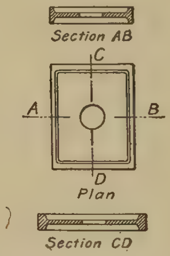


FIG. 46*d*.—Cast Brace Block.

Deepened beams are less frequently employed in this country than abroad, but they may be found in temporary and other timber structures for mining operations remote from large cities, as well as in railroad beam bridges in localities where timber is cheaper and more accessible than steel.

The following statement regarding deepened beams is made by J. P. Snow in an article on Modern Bridge Construction on the Boston and Maine Railroad: "These stringers require considerably more material than trusses of equal strength, but the labor on them is small and they can be put in place and prepared for the passage of trains in much less time than trusses. This latter quality is of great importance, and should be given more attention by bridge designers than it generally receives. This style of bridge works in with the ordinary trestle spans very conveniently when it is desired to make a wide opening for a runway for ice or logs, or for a highway underpass. The lower stick of the compound stringer is extended beyond the upper one to furnish a seat for the regular stringers of adjacent bents. In cases, too, where the trestle bents are high and expensive, it will lessen the cost of the structure to make alternate spans compound or keyed stringers. This style of bridge is available up to clear spans of 30 ft."

PROBLEM 46. Refer to Warren's Engineering Construction in Iron, Steel, and Timber, page 92, and copy the illustration of a deepened beam of ironbark wood in which the keys vary in size and are spaced uniformly instead of being uniform in size and spaced unequally.

ART. 47. PRINCIPLES GOVERNING DESIGN

In the design of a deepened beam the following principles and details should be considered, in order to secure the same degree of safety in all its parts as well as economic construction.

Relation of Width to Depth of Beam.—At or near the section where the bending moment is a maximum the strength depends upon the stress in the outer fibers, and is therefore proportional to the width and to the square of the depth. At the ends of the beam the strength depends on the unit stress in horizontal shear at the neutral surface, and hence the strength varies directly as the width and depth. Accordingly, the most economical width is that which gives only sufficient shearing area between the two keys having the least spacing. As, however, the end key may be moved beyond the support without increasing the shear to be resisted by it, the spacing between the second and third keys really determines the relation of width to depth.

Since the length of shearing surface required can only be found after a preliminary determination of the cross-section, the width may be assumed for this purpose as equal to, or a little greater than, the half depth. Instead of assuming the width b, and finding the depth d after the value of bd^2 is known, it is better to assume d first and then find b, revising the result until the dimensions have the relation indicated above.

Number of Keys.—The number is to be assumed so as to give a reasonable depth to provide the necessary bearing area on the ends of the keys. It is to be remembered that the cost of workmanship increases with the number of keys, while the difference between the cost of a small and of a large key or brace block is relatively slight. As the number of keys is increased it also requires better workmanship to have them all act together to take their proper share of the horizontal shear. On the other hand, as the number of keys is reduced, the distances are increased through which the stresses must be transmitted from one timber to the other.

Design of the Brace Blocks.—The web of the brace block transmits the resultant compression in its direction from one end flange to the other. The flanges at the sides serve mainly to stiffen the web. The flanges at the ends act as double cantilevers supported by the web, and receive as a uniformly distributed load the compression or bearing of the fibers of the beam. Sometimes the thickness of the web is governed by the minimum thickness allowed by practical considerations in founding rather than by the stress transmitted. The flanges must have a slight bevel on the inside to allow the pattern to be readily withdrawn from the mold. Since the resultants of the horizontal pressure on the ends of the block are eccentric, the block tends to rotate, and this tendency to rotation must be resisted by the vertical pressure of the sides of the fibers on the end flanges. In order to keep the length of the brace block within reasonable limits, it may be necessary to thicken the flanges to increase this bearing area. To keep the timbers from separating they are bolted together through the block, the tension in the bolt being equal to the resultant of the vertical pressure on each end of the brace block. If the beam is exposed to the weather, the blocks may be made narrower than the timber, thus preventing water from collecting in the notches, or the beam may be protected by a covering of tin properly arranged. In general the length of the key should be nearly equal to its width.

Position of the Brace Blocks.—Each block can resist only horizontal shear which is developed on its side toward the section where the maximum bending moment is located. The blocks are therefore to be placed with their inner edges at the points which divide the total horizontal shear into equal parts. An error in design which is often noticed consists in placing the end block inside of the center of the support. As the distance between the inner blocks of each half span is relatively large, it is usually necessary to keep the timbers in close contact by additional bolts, taking care not to locate any bolt so close to the section of maximum bending moment as to increase the unit stress beyond its safe value.

Length of the Shearing Surface.—When the keys or blocks are horizontal, as in Fig. 17*a*, there is no doubt as to the length of available shearing surface between the blocks; but when they are inclined, it depends upon the direction of the grain in the timber. It may be close enough for practical purposes to assume that the shearing surface for any block extends from its outer end to the middle of the next block. Since no vertical shear exists beyond the support of a simple beam, no horizontal shear can develop there, and hence the shearing surface between the first and second blocks may be increased if desired by moving out the end block. If any other block be moved, it changes the pressure on it and on the next outer block.

On account of the shrinkage of the timbers as seasoning progresses, and the tendency of the brace blocks to separate them, no reliance can be placed upon the friction between the two timbers to resist any part of the horizontal shear.

ART. 48. DESIGN OF A DEEPENED BEAM

The following order of design is suggested for the guidance of the student: 1. Draw the symbolic loading in position. 2. Construct the bending moment diagram for the external loading on the beam. 3. Construct the vertical shear diagram for the external loads. 4. Find the dimensions of a solid timber to resist the greatest bending moment, remembering the relation between width and depth indicated in Art. 47. 5. Compute the weight of the timber and estimate the additional weight of details, the opposite sides of the axes of the bending moment and vertical shear diagrams; construct the corresponding diagrams due to the weight of the beam. 6. Revise the section of the beam, if its resisting moment is not great enough to include the bending moment due to its own weight. 7. Compute total horizontal shear. 8. Assume the number of keys or brace blocks in the half span, and compute the horizontal shear to be resisted by each block. Find the position of the inner sides of the blocks. 9. Compute the length of shearing surface required for each block. 10. Design a brace block to resist this pressure: (*a*) width of end flange; (*b*) thickness of web; (*c*) thickness of end flanges at the edge; (*d*) their thickness next to web; (*e*) length of the brace block; (*f*) thickness of side flanges. 11. Design the bolts and washers. 12. Compute the bearing areas and widths required for the supports.

For example, let it be required to design a deepened beam for indoor use with a span of 20 ft., to support two concentrated loads, one of 6000 lb. at 7 ft. and the other of 15,000 lb. at 15 ft. from the left support, and also a uniform load of 5000 lb. per lin. ft. extending between two points respectively distant 3 and 13 ft. from the left support.

The following working unit stresses are specified:

Dense-Select Douglas Fir

	Pounds per Square Inch
Extreme fiber stress in bending	1,750
Longitudinal shear	210
Compression on the sides of the fibers	380
Compression on the ends of the fibers	1,710

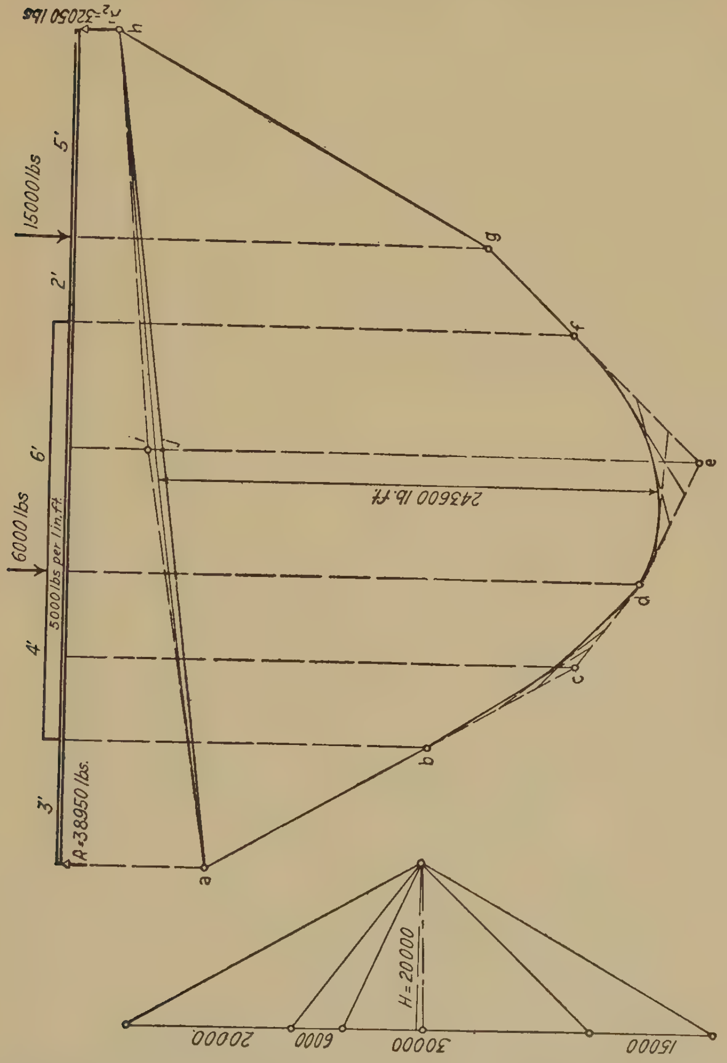
Cast-Iron Brace Blocks

Extreme fiber in bending	5,000
Compression	15,000

Steel Bolts

Tension	16,000

The loading is indicated in Fig. 48a in magnitude and position, a scale of 2 ft. to an inch being used for the original drawing. The force polygon and equilibrium polygon are constructed in accordance with the principles of graphic statics. A convenient scale for the load line is 10,000 lb. to an inch. The pole distance is made 2 in., thus making H = 20,000 lb. In drawing the equilib-



R2=32050 lbs
h
5'
15000 lbs
2'
g
f
6'
i
j
e
243600 lb.ft
6000 lbs
5000 lbs per lin.ft.
d
4'
c
b
3'
R1=38950 lbs.
a
H=20000
20000
6000
30000
15000

FIG. 48a.—The Graphical Analysis of a Deepened Beam.

rium polygon, the uniform load on each side of the 6000-lb. concentrated load is concentrated at the two respective centers of gravity, giving the polygon *acdegh*, and then the two parabolas are drawn tangent to the sides of the polygon at the sections where the corresponding portions of the uniform load begin and end. To construct the parabola *df*, divide both *de* and *ef* into the same number of equal parts and join corresponding points by straight lines as shown in the illustration. The parabola will be tangent to these lines.

The ordinates y intercepted between the rays of the equilibrium polygon are measured by the linear scale, and these, when multiplied by H, give the moments at various sections of the beam, since by graphic statics the moment $M = Hy$. If preferred, the moments may be read off directly by using a scale of 40,000 ft.-lb., this moment scale being H times the linear scale. The greatest bending moment scales 237,000 ft.-lb. For a unit stress of 1750 lb. per sq. in. the resisting moment of a beam of breadth b and depth d is 1750 $bd^2/6$ in.-lb. Equating the value of the two moments $bd^2 = 9750$ in.^{3} Assuming $d = 28$ in., b is found to be 12.44 in. A value of 14 in. will be used, which allows for a 1-in. bolt hole. At 42 lb. per cu. ft. the weight of the beam is 2290 lb., and in order to make ample allowance for bolts and brace blocks the total weight is assumed to be 2600 lb. The bending moment at the middle of the span due to this dead load is $\frac{1}{8} \times 2600 \times 20 = 6500$ ft.-lb. The parabolic bending moment diagram for this dead load can be placed on the diagram of Fig. 48*a* by laying off $ji = 2 \times 6500 = 13{,}000$ ft.-lb. to the scale of 40,000 ft.-lb. to the inch, drawing the lines *ai* and *hi*, and then dividing these lines into the same number of equal parts and proceeding as outlined above for the live-load parabolic curves.

The total moment at any section is given by the ordinate between the two curves, the scale being 1 in. = 40,000 ft.-lb. The maximum moment is 243,600 ft.-lb. and occurs at 9.35 ft. from the left support. These values can be checked analytically, the maximum moment occurring where the shear equals zero. The left reaction is 38,950 lb. The 14- by 28-in. section will take a moment—allowing for a 1-in. bolt hole—of $1750 \times 13 \times 28^2/(6 \times 12) = 247{,}500$ ft.-lb., hence no revision of section is necessary.

The total horizontal shear along the neutral axis of a beam from the end to any vertical section equals 3M/2*d*, since it equals the total compression or tension on the vertical section. The compression and tension stresses form a couple whose moment equals the moment of the external forces and whose moment arm is 2*d*/3. In this problem the total horizontal shear from the end of the beam to the section of maximum moment is 3 × 243,600 × 12/(2 × 28) = 156,000 lb. Assuming four brace blocks are used on either side of the center, each one resists a compression due to horizontal shear of 156,600/4 = 39,150 lb.

Since the total horizontal shear from the end of the beam to any vertical section is proportional to the moment at that section and since the ordinates of the equilibrium polygon of Fig. 48*a* give values of M when a scale of 1 in. = 40,000 ft.-lb. is used, it follows that to some scale these ordinates represent the total horizontal shear. Therefore the locations of the four blocks may be found by finding the positions of those ordinates having values one-quarter, one-half, and three-quarters those of the maximum ordinate. On the left side these distances are found to be, respectively, 1 in., 6¾ in., 3 ft. 2 in. and 5 ft. 0 in. from the center of the left support. The corresponding sections on the right portion of the beam are located 1 ft. 9 in., 3 ft. 6½ in. and 5 ft., 7¼ in., respectively, from the right support. Since the brace block at the left support may be moved farther to the left if necessary, without changing the pressure which it must resist, the critical spacing is that between blocks 2 and 3. The distance between the sections is 38−18.75 = 19.25 in. For an allowable unit stress of 210 lb. per sq. in., the net length of shearing surface is 39,150/(210 × 14) = 13.3 in. The difference between these distances is probably sufficient to allow for the notch cut for the block and some lateral deviation of the grain of the timber.

The height of the brace block or width of its end flanges must provide sufficient bearing area for compression on the ends of the wood fibers. For a unit stress of 1710 lb. per sq. in., the width of flanges is 39,150/(1710 × 14) = 1.64, or 1¾ in. Allowing for a 2-in. hole in the web, its thickness must be at least 39,150/(15,000 × 12) = 0.22 in.; but to secure a good casting of the size of such a block, the web should not be less than ½ in. Each cantilever of the flange projects ½(1.75 − .50) = 0.625 in. beyond the web, and its load is 39,150 × 0.625/1.75 = 13,980 lb. The maximum bending moment is in the section at the web, and equals 4369 in.-lb. Equating this to the resisting moment of the section of the cantilever, the working unit stress in the cast iron being 5000 lb. per sq. in., the thickness of the flange required is found to be 0.612 or $\frac{5}{8}$ in.

Let the length of the block (along the beam) be assumed as 12 in., and let a width of $1\frac{1}{4}$ in. be assumed for the bearing at the flanges to resist rotation, then a sketch of the side elevation of the block indicates a lever arm for the bearing couple of 10.75 in. (Fig. 48*b*). The moment of rotation for the block is 39,150 × 1.75 = 68,510 in.-lb., and the pressure on each side bearing is 68,510/10.75 = 6370 lb. For a unit stress of 380 lb. for compression on the side of the fibers, the width of bearing required is 6370/(380 × 14) = 1.20, or $1\frac{1}{4}$ in. Hence the thickness of flange at the outside will be made $1\frac{1}{4}$ in. and at the web $1\frac{3}{8}$ in., thus allowing the pattern to be easily drawn from the mold. The side flanges will be made $\frac{3}{8}$ in. thick at the outside and $\frac{1}{8}$ in. thicker at the web.

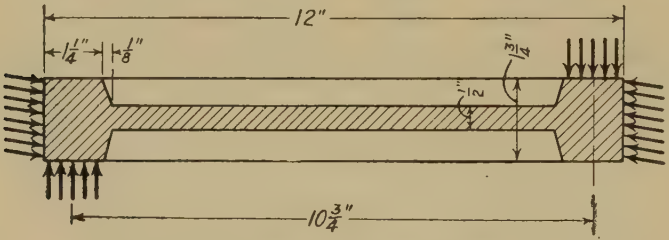


FIG. 48*b*.—Pressures on Brace Block.

The stress in the bolt is practically equal to 6370 lb., the compression on the side of the fibers. Hence the area required at the root of the thread is 6370/16,000 = 0.398 sq. in. A diameter of $\frac{7}{8}$ in. will be used, giving an area of 0.419 sq. in. The washer requires a diameter of 5 in., which furnishes a net area of 18.8 sq. in., to provide the needed bearing area of 6370/380 = 16.75 sq. in.

The bearing area required for the left support of the beam is 38,950/380 = 102.5 sq. in.; hence the length along the beam is 102.5/14 = 7.32 in. A length of 10 in. will be used, since the washer cuts out some of the bearing area. The other bearing is practically the same. Let brace block 1 be moved $\frac{1}{2}$ in. to the left, in order to provide the same spacing between block 1 and block 2 as between blocks 2 and 3.

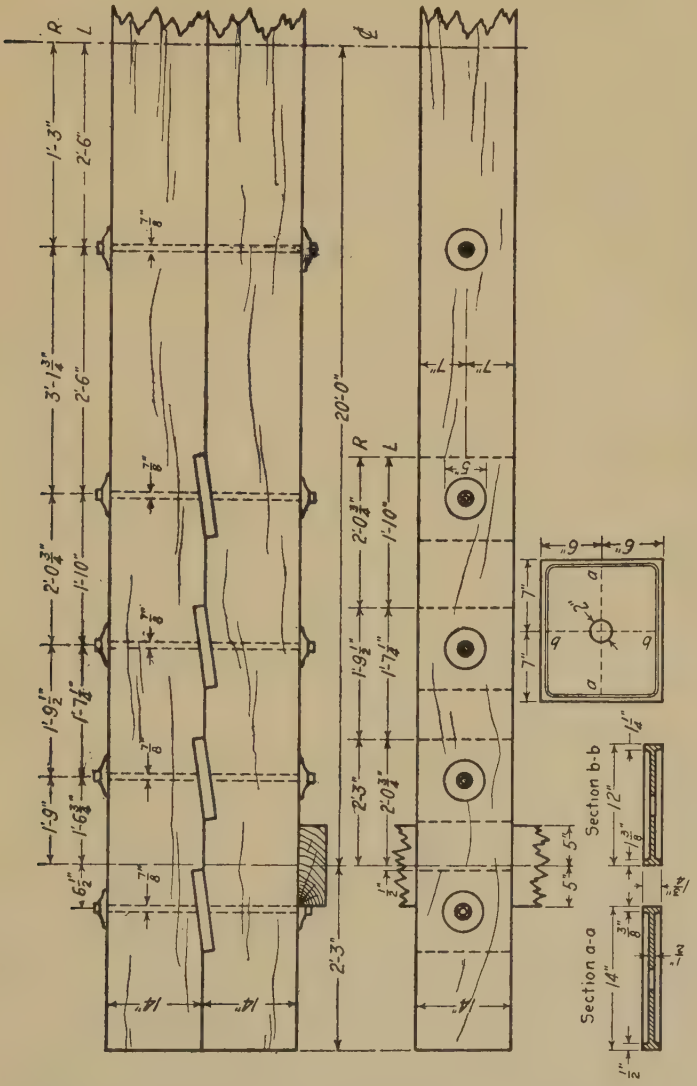

R
L
℄
1'-3"
2'-6"
3'-1¾"
2'-6"
2'-0¾"
1'-10"
1'-9½"
1'-7¼"
1'-9"
1'-6¾"
6½"
7/8"
20'-0"
2'-0¾"
1'-10"
1'-9½"
1'-7¼"
2'-3"
2'-0¾"
½"
2'-3"
14"
14"
14"
7"
7"
5"
6"
6"
7"
7"
2"
a
b
5"
5"
Section b-b
12"
1⅜"
¼"
¾"
Section a-a
14"
⅜"
¾"
½"

FIG. 48c.—Working Drawing of Deepened Beam and of the Cast-iron Brace Block.

The detail drawing of the left half of the beam, with all the required dimensions for the entire beam, as well as that of the cast brace block, is given in Fig. 48c. The bill of material is as follows:

2 timbers, 14 × 14 in. × 24 ft. 6 in. (26 ft. 0 in.)	849.3 ft. b.m.
8 cast-iron brace blocks, 12 × 14 in.	295.0 lb.
10 $\frac{7}{8}$-in. bolts, $31\frac{1}{2}$ in. long	57.5 lb.
20 O. G. washers, 5 in. in diameter	64.0 lb.

The estimate of cost is as follows:

850 ft. b.m. lumber @ 6 cents	$51.00
295 lb. cast iron @ 5 cents	14.75
58 lb. steel bolts @ 6 cents	3.48
64 lb. cast-iron washers @ 5 cents	3.20
18 hrs. labor @ 80 cents	14.40
	$86.83

Attention is called to the fact that large timbers like those included in the above bill of material are usually sawed to order, in which case they can be cut to the length desired, thus saving somewhat on the cost.

The gross weight of the lumber for a length of 24 ft. 6 in. is 2800 lb., and the weight of the brace blocks, bolts and washers is 416 lb., or the weight of the details is 14.8 percent of the main timbers. For the effective span of 20 ft. the weight is 2630 lb., which is 30 lb. more than the dead load assumed.

Problem 48. Compute the size of a solid beam of minimum weight, to carry the same loads as the deepened beam used in the previous example, under the assumption that the beam is a single stick, sawed to order, to within a quarter of an inch of each dimension required.

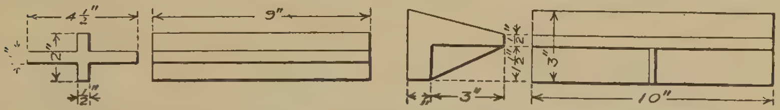
Art. 49. Construction of Deepened Beams

In the construction of deepened beams only timber which is straight-grained should be employed. If in any part of the length the grain is not quite parallel to the horizontal surface, especial care should be taken to cut the notches or indents on that side where the grain runs away from the surface toward the ends. Unless this precaution is observed, the shearing surface between the keys may be shorter than the length designed.

The seats of the brace blocks in the two timbers are to be so located with respect to each other that when the parts are put together the beam will have a camber. The initial camber must be sufficient to allow for the local adjustment, which brings all the blocks to a solid bearing, and also to prevent the beam from deflecting below the horizontal when under its full load. This is necessary to insure that all the fibers of the lower stick shall be in tension.

Before fastening the timbers together their adjacent surfaces should be painted with hot creosote oil, avenarius carbolineum, or some other preservative in case the beam is to be exposed to the weather (Art. 38). The bolts should be drawn up to their safe stress so that no further elongation occurs when the beam is loaded. If the timber is not seasoned, the bolts will require adjustment afterward.

Figure 49*a* shows another form of key which has been used in practice. The compression of the horizontal plates on the sides of the fibers varies from zero at the center to the safe unit stress at the ends. The four arms act as cantilevers, and the thickness of each one near the middle must resist the maximum bending moment. While cast-iron keys of this form have been employed in deepened beams used as railroad stringers, it would be safer to use cast-steel keys, since the flexural strength of cast iron is more or less unreliable. The main advantage of this form consists in reducing the loss of horizontal shearing surface to a minimum. Figure 49*b* gives the form of key used on the Boston & Maine Railroad.



FIGS. 49*a* and *b*.—Special Forms of Keys for Deepened Beams.

A beam composed of a single timber contains the minimum amount of wood when it is designed so that both the greatest unit stress in the outer fibers and that in horizontal shear at the neutral surface are respectively equal to the allowable safe values. Any deepened beam of the same strength and kind of wood requires more material, since its width must be greater in proportion to its depth on account of the loss of shearing surface due to the keys.

When metal keys are employed and the beam is constructed with a camber, its stiffness is practically the same as for a single stick having the same outside dimensions. When, however, wooden keys are inserted with the grain across the beam, even when tightened with wedges, it is impracticable to secure the same stiffness as with metal keys, since the wooden keys will compress appreciably when the full load is applied. Any movement of the contiguous surfaces of the timbers increases the deflection. Experiments show that with wooden keys the fibers should be placed parallel to those of the beam. Below the elastic limit there is no danger of the fibers interlacing.

The results of an elaborate series of tests of deepened beams with clear spans of 66 and 90 in. may be found in an article on The Efficiency of Built-up Wooden Beams, by Edgar Kidwell, in Transactions of American Institute of Mining Engineers, vol. 27, pages 732–818, with discussions on pages 970–993. An abstract was published in Engineering News, vol. 37, page 149; and vol. 39, pages 76, 182.

Art. 50. Trussed Beams or Girders

When the span is too large for the economic use of a single wooden beam, a trussed beam is sometimes substituted when conditions permit a larger depth of space to be occupied. The simplest form is illustrated in Fig. 50*a*, where the beam is supported by a single strut or king-post at the center. To avoid secondary stresses the center lines of beam, rod, and support must intersect in a point. Two tie-rods are frequently used which support the strut and pass outside of the beam, with nuts bearing on a thick plate of steel at each end of the beam.

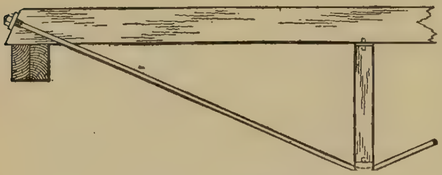


Fig. 50*a*.—Trussed Beam.

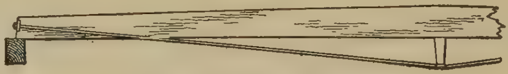


Fig. 50*b*.—Trussed Beam.

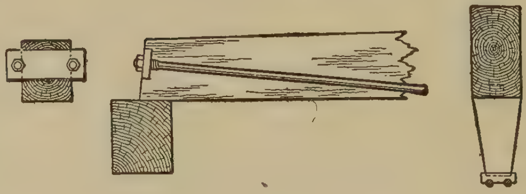


Fig. 50*c*.—Enlarged Details.

The beam is generally of uniform cross-section, but sometimes it is deeper at the middle as in Fig. 50*b*. In this figure the strut is relatively very short, thus making the beam much less effective. Small beams are sometimes composed

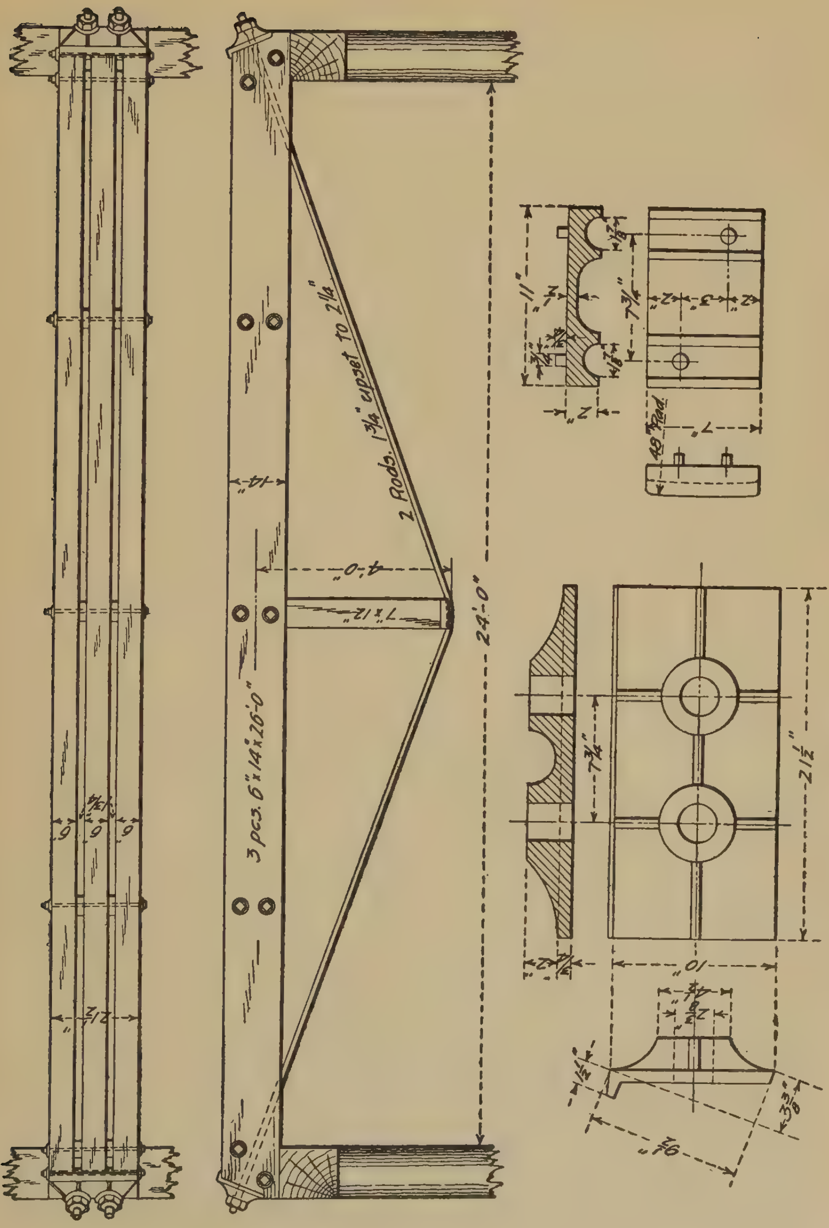

3 pcs. 6"×14"×26'-0"
2 Rods. 1¾" upset to 2¼"
4'-0"
7"×12"
24'-0"
14"
6
6
6
12"
11"
7¼"
7¼"
21½"
10"
7"
9⅞"
3⅜"
49" Rad.

FIG. 50d.—Trussed Beam on One Side of a Railroad Beam Bridge.

of two timbers with a single truss rod passing between their ends. In this case the strut is conveniently fastened to the horizontal timbers by a tenon passing between them and connected with bolts or wooden pins. When two rods are used, the beam may be divided into three sticks, thus allowing the rods to pass between them. This arrangement is shown in Fig. 50*d*. The cast-iron bearing plates and other details indicate that they were carefully designed.

When the trussed beam or girder supports a floor which effectively braces the sticks laterally, the strength is practically the same as if only a single stick is used. When, however, only a concentrated load is supported directly above the vertical strut, the two timbers will act as though they were entirely disconnected between the load and the supports, even though filler blocks may be located at intervals and the timbers bolted together. The unit stress in compression must then be reduced as in columns for the ratio of the length to the least dimension or least radius of gyration, depending upon the formula adopted.

When two concentrated loads are to be supported, as in the longer spans for which trussed beams are adapted, two vertical struts of queen-posts are used as illustrated in Fig. 50*e*. The figure shows a cast-iron strut of star-shaped cross-section. The most common example of the queen-post trussed beam is found in the trussed sills of wooden car frames, illustrations of which may be seen in the Car Builders' Dictionary, or in the volumes of Railroad Age Gazette and other railroad periodicals. In car construction the ends of the rods are flattened and bolted to the sill, and a turnbuckle inserted at the middle of the span to tighten up the rods, or to secure the necessary camber (see Fig. 50*f*).

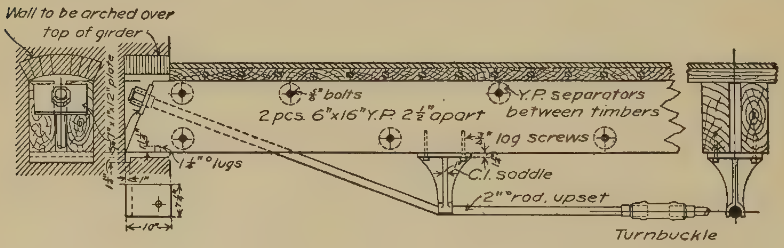


FIG. 50*e*.—Details of Trussed Floor Beams for Warehouse.

In the queen-post form the struts are sometimes inclined so as to bisect the angle between the horizontal and inclined portions of the truss rod in order to avoid any tendency to flexure. A beam with this detail used as a floor beam in a conveyor bridge is illustrated in Engineering Record, vol. 34, page 159, August 1, 1896. The span is 18½ ft., the beam is composed of two sticks

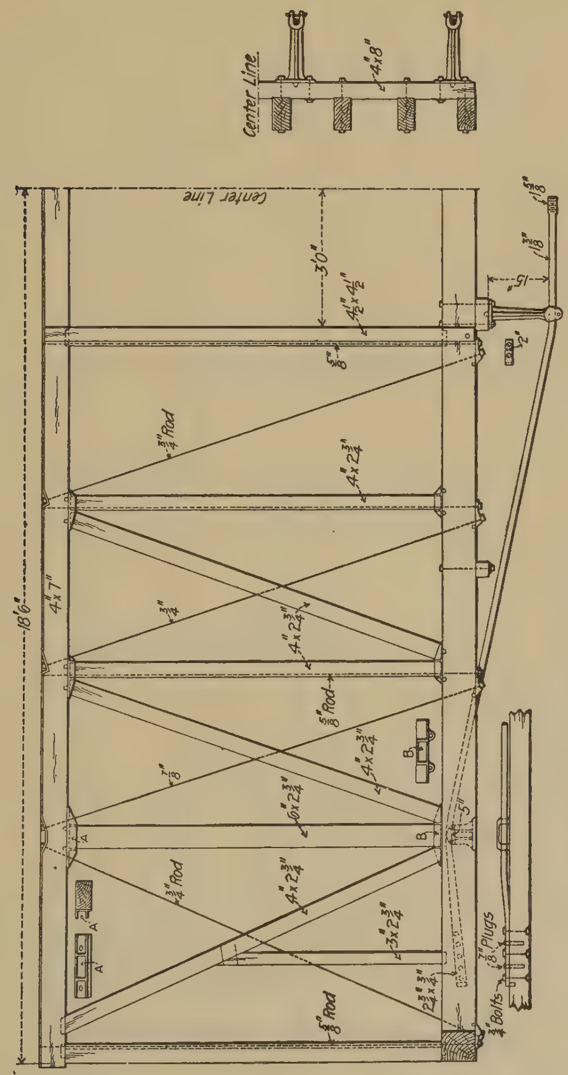

Center Line
4"x8"
Center Line
18'6"
4"x7"
3'0"
4½"x4½"
5/8"
2"
15"
18"
3/4" Rod
4"x2¾"
3/4"
4"x2¾"
5/8" Rod
7/8"
6"x2¾"
4"x2¾"
B
B
A
B
5"
3/4" Rod
4"x2¾"
3"x2¾"
A
A
5/8" Rod
2¾"x4"
7/8" Plugs
3/4" Bolts

FIG. 50*f*.—Kirby's Framing for a Box Car of 80,000 Pounds Capacity.

This drawing shows the application of truss rods to strengthen and stiffen the sills of the underframe. "The design provides ample supports for the four truss rods 1⅜ inches in diameter over the bolsters and needle beams. The ends are flattened to 2¾ by ¾ inch and secured to the longitudinal sills by two ⅞-inch iron plugs and three ¾-inch bolts, making a strong and simple fastening." *Railroad Gazette*, vol. 34, page 961, Dec. 19, 1902.

4 by 12 in. spaced $2\frac{1}{4}$ in. apart in the clear, and each strut is 6 by 6 in. At the upper end a tenon projects between the timbers and is connected by two bolts $\frac{5}{8}$ in. in diameter, and at the lower end is a cast bearing block with a projecting dowel. Only one rod is used with a diameter of $2\frac{1}{8}$ in.

A novel application of the trussed-beam construction is described in an article on An Ingenious Method of Hauling Heavy Machinery over Country Roads, in Engineering News, vol. 51, page 62, January 21, 1904.

An interesting comparison between the strength and deflection of trussed beams, flitch beams, and a combination of the two, with a plain wooden beam and a pressed-steel beam, is recorded in Railroad Gazette, vol. 27, page 50. The beams tested are in the form of car bolsters, supported near the ends and loaded at the center.

Art. 51. Design of Trussed Beams

On account of the continuity of the beam from one support to the other, the true method of determining the stresses in all the members of a trussed beam requires the aid of the principle of least work. If a single load is placed at *B* in Fig. 51*a*, some part of it will be carried by the timber *AC* acting as a simple beam; and the rest of the load will be carried by the truss of which the beam forms the upper chord. According to the principle of least work, the load is divided so as to make the total internal work of all the stresses in the members a minimum. It also follows that the division of the load is such as to cause the same deflection of the point *B* in the simple beam as in the truss.

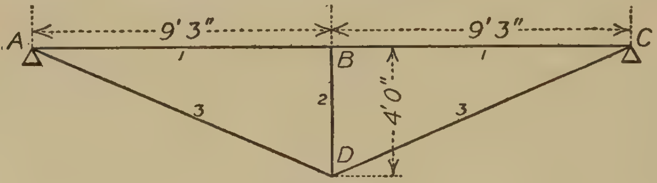


Fig. 51*a*.

To find the stresses by means of the method of least work or by the method of deflections requires that the sectional areas of the members must be known. It is therefore necessary to find these approximately, and then to investigate the structure to learn if any of the stresses exceed the specified limit.

For example, let it be required to design a trussed beam for a building, of the span and depth indicated in Fig. 51*a*, to support a uniformly distributed load, including its own weight, of 1800 lb. per lin. ft. The beam is held in position laterally by the flooring. With the aid of a table of squares (Art. 92), the length of the diagonal AD or DC is found to be 10.078 ft. or 10 ft. $0\frac{15}{16}$ in. If θ is the angle which the diagonal makes with the vertical, $\tan \theta = 9.25/4 = 2.3125$ and $\sec \theta = 2.5197$. Let it be assumed that the beam AC is fully continuous, that is, that the point B does not deflect. In this case the stress in BD would equal the reaction at B of the continuous beam AC, which, for a uniform load, is $\frac{5}{8} \times 1800 \times 18.5 = 20{,}810$ lb. Taking the joint D as a free body, the tension in the diagonals is found to be $\dfrac{20{,}810}{2} \times \sec \theta = 26{,}220$ lb. The compression in the upper chord is $\dfrac{20{,}812}{2} \times \tan \theta = 24{,}060$ lb.

The left reaction of the beam AC is $\frac{3}{16} \times 1800 \times 18.5 = 6244$ lb., and the moment at B is $6244 \times 9.25 - 16{,}650 \times 4.625 = 19{,}250$ ft.-lb.

The beam and strut are to be made of dense-select Douglas fir, with the following allowable unit stresses expressed in pounds per square inch:

Extreme fiber stress in flexure, 1750; compression on the side of the fiber, 380; end compression on columns under 10 diameters, 1285; and modulus of elasticity, 1,200,000. This modulus is less than the value given in Art. 89, since part of the load is fixed or permanent. The diagonal rods are to be of structural steel, the allowable tensile stress being 16,000, and the modulus of elasticity 29,000,000.

Let a 6 by 12-in. beam be assumed. The unit stress due to flexure is $\dfrac{6 \times 19{,}250 \times 12}{6 \times 12 \times 12} = 1605$ lb. per sq. in. The unit stress due to direct compression is $24{,}060/72 = 335$, the sum being 1940. This is greater than the safe value given above but the exact analysis always results in a lower value as the point B deflects somewhat.

The section area needed by the strut BD is $20{,}810/1285 = 16.7$

sq. in. But the bearing of the strut on the bottom of the beam really governs the size unless a bearing plate is provided. The bearing area required is 20,810/380 = 54.7 sq. in.; hence a 6 by 9-in. strut will be adopted.

The required section area of the tie-rods is 26,220/16,000 = 1.637 sq. in. On account of the length, we will use rods with upset ends, hence the diameter required is $1\frac{1}{2}$ in., providing a sectional area of 1.767 sq. in.

Having chosen tentative sections for all members, the true stresses may now be found. Let the beam be imagined separated from the truss by an incompressible block BB' at B. The loads on the beam AC will then consist of a downward uniform load of 1800 lb. per foot of length and of an upward concentrated load at B—the stress in BB'—which we will call P. The load on the truss will consist of a downward load P at B'.

The deflection at the center of a beam under a uniform load is $\frac{5}{384}\frac{Wl^3}{EI}$ and under a concentrated force at the center $\frac{1}{48}\frac{Pl^3}{EI}$. Hence for the beam in question the deflection in inches is

$$\frac{5 \times 33{,}300 \times \overline{222}^3}{384 \times 1{,}200{,}000 \times 864} - \frac{P \times \overline{222}^3}{48 \times 1{,}200{,}000 \times 864} = 4.575 - .0002198P$$

The deflection at B' of the truss may be found by means of the expression $\Sigma \frac{STL}{AE}$, where S is the stress in a member due to the load P, T the stress due to a load unity at B', A the sectional area of the member and E the modulus of elasticity. In this case $S = PT$.

Member	T Pounds	L Inches	A Square Inch	E Pounds Per Square Inch	$\frac{PT^2L}{AE}$
Strut $B'D$...	−1	48	54	1,200,000	0.000000740
Chord AC...	−1.156	222	72	1,200,000	0.000003433
Tie AD.....	+1.260	120.9	1.767	29,000,000	0.000003746
Tie DC......	+1.260	120.9	1.767	29,000,000	0.000003746

The sum of the products in the last column is 0.000010665.

The imaginary member BB' was taken as incompressible—since the points B and B' are actually one and the same—and hence $4.575 - 0.0002198P = 0.00001066\,P$, or $P = 19{,}850$. The deflection at B equals $.00001066 \times 19{,}850 = 0.212$ in.

The two end reactions of the beam are $\frac{1}{2}\,(33{,}300 - 19{,}850) = 6725$ lb. and the moment at B is $6725 \times 9.25 - 16{,}650 \times 4.625 = -14{,}800$ ft.-lb. The maximum positive moment occurs where the shear equals zero. Calling this distance from the left support x, then $1800\,x = 6725$, or $x = 3.736$ ft. The moment at this section is $6725 \times 3.736/2 = 12{,}560$ ft.-lb.

The tension in the diagonals is $19{,}850 \sec \theta/2 = 25{,}000$ lb.; the compression in the strut is 19,850 lb. These values are so close to those used in the preliminary design that no revisions of sections are necessary. The direct compression in the upper chord is $19{,}850 \tan \theta/2 = 22{,}950$ lb. The maximum bending stress in the beam is $\dfrac{6 \times 14{,}800 \times 12}{6 \times 12 \times 12} = 1233$ lb. per sq. in.; the unit direct compression is $22{,}950/(6 \times 12) = 319$ lb., making the total stress 1552 lb. per sq. in. The deflection of 0.212 in. at B results in an added moment of $22{,}950 \times 0.212 = 4860$ in.-lb., resulting in an added stress of 34 lb. per sq. in. The total unit stress is well within design limits.

In some cases it may be necessary to build the trussed beam with an initial camber. For example, let a camber of $\frac{3}{16}$ in. be put into the beam under consideration in this article. This camber may be obtained by buckles shortening the rods by the use of turnbuckles. With a $\frac{3}{16}$-in. camber the deflection of the beam at B will be $0.212 - 0.1875 = 0.0245$ in. The deflection of the beam is given by the expression $4.575 - 0.0002198P$. Equating this to 0.0245 in., $P = 20{,}700$ lb. Hence the total stress in the rods is $\dfrac{20{,}700 \sec \theta}{2} = 26{,}100$, and the unit stress is $26{,}100/1.767 = 14{,}770$ lb. per sq. in. The unit stress between the strut and the beam is $20{,}700/(6 \times 9) = 383$ lb. per sq. in. The moment in the beam at b is $33{,}300 \times 18.50/8 - 20{,}700 \times 18.50/4 = -18{,}730$ ft.-lb. Hence the bending stress is $6 \times 18{,}730 \times 12/(6 \times 12 \times 12) = 1560$ lb. per sq. in. The axial compression is $\dfrac{20{,}700 \tan \theta}{2} =$

23,930, and the unit stress is 23,930/(6 × 12) = 330, making the total unit stress 1890 lb. per sq. in.

The handbook, Southern Pine Manual, 1927, contains typical trussed-beam designs for spans from 16 to 22 ft. for the king-post type and from 20 to 30 ft. for the queen-post type, uniform loading being used. For the first type the tension in the rods is taken at $0.3\ W\ \sec\theta$, the compression in the strut at $0.3\ W$, and the compression in the beam at $0.3\ W\ \tan\theta$. The above example indicates that these are close approximations. The bending moment in the beam is determined on the assumption that each half of the span acts as a simple beam carrying that part of the load on it. In the above example the moment would be 33,300 × 9.25/(2 × 8) = 19,250 ft.-lb., which is considerably above the true value of 14,800 ft.-lb.

For the queen-post type the tension in the rods is taken at $0.376\ W\ \sec\theta$, the compression in the struts at $0.376\ W$ and the compression in the beam at $0.376\ W\ \tan\theta$. The bending moment in the beam is found on the basis that one-third of the span acts as a simple beam carrying the load on it. The student should check the validity of these assumptions by designing a truss by the exact method. The deflection at the third points of a beam under a uniform load is $\frac{11Wl^3}{972EI}$, whereas for two concentrated loads at the third points the deflection at the same points is $\frac{13Pl^3}{648EI}$ where P = each of the two loads.

ART. 52. DESIGN OF WOODEN COLUMNS

According to full-size tests, wooden columns generally fail by lateral deflection or buckling, when the length exceeds about twenty times the least dimension of the cross-section. In practice, however, the unit stress in all columns with length over ten or fifteen diameters should be reduced in accordance with various rules or formulas established for long columns. The same unit stress is employed for all shorter columns as for those having a slenderness ratio of $\frac{l}{d} = 10$ or $\frac{l}{d} = 15$, in which l is the length and d is the least dimension or diameter, both being expressed in inches.

Since most wooden columns are rectangular or square in section, it is customary to employ column formulas containing the ratio $\frac{l}{d}$ rather than the more general form containing the ratio $\frac{l}{r}$, the term r denoting the least radius of gyration of the cross-section, The data which are given when a column is to be designed consist of the axial load, its length and the conditions of the ends. The column formula to be used is either given in the specifications or must be selected by the designer. Two column formulas widely used are given in Art. 90. No column should have a ratio of $\frac{l}{d}$ greater than 50.

As the sectional area depends upon the average unit compressive stress, and this in turn depends upon the ratio of the length to the least dimension of the cross-section, an approximate sectional area must first be found in order to estimate closely the least dimension of the column. For example, let it be required to design a rectangular column 10 ft. long, of dense-select Douglas fir for in-door use, to support an axial load of 28,450 lb. Assuming the least dimension will be $5\frac{5}{8}$ in., $\frac{l}{d} = 10 \times 12/5.62 = 21.3$. From Table 90*a*., the value of K is found to be 22.6, hence the formula for intermediate-length columns,

$$\frac{P}{A} = S\left[1 - \frac{1}{3}\left(\frac{l}{Kd}\right)^4\right],$$

is applicable.

From the same table the value of S is found to be 1285, and from Table 90*b* we find that $\left[1 - \frac{1}{3}\left(\frac{l}{Kd}\right)^4\right]$ is 0.725. Hence $\frac{28,450}{A} = 1285 \times 0.725$, or the required area is 31.0 sq. in. Hence a 6- by 6-in. section, dressed to $5\frac{5}{8}$ by $5\frac{5}{8}$ in., giving an area of 31.6 sq. in., will be adopted. No correction is necessary, since the correct minimum dimension was assumed.

Designing the same column by the formula

$$\frac{P}{A} = 1800\left(1 - \frac{l}{60d}\right),$$

as given in Table 89*b*, assuming $d = 5\frac{5}{8}$ in.,

$$\frac{28{,}450}{A} = 1800\left(1 - \frac{10 \times 12}{60 \times 5.62}\right),$$

which gives a required area of 24.5 sq. in. Hence a $5\frac{5}{8}$ by $5\frac{5}{8}$-in. section is required.

Art. 53. Construction of Posts

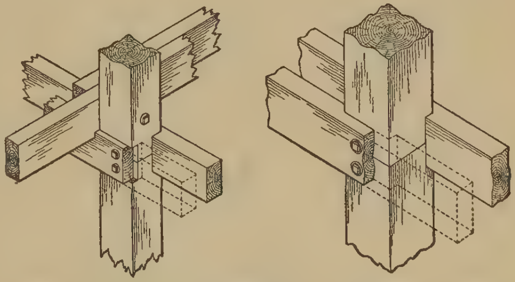
To reduce the tendency of large timbers to check when used as posts in buildings, it is customary in many cases, especially in slow-burning or mill construction (Art. 75) to bore a $1\frac{1}{2}$-in. hole longitudinally through its center, and to provide for the circulation of air through it by means of $\frac{1}{2}$-in. vent holes at the top and bottom. Especial care should be exercised to secure perfectly square ends for bearings in order to avoid secondary bending moment due to the eccentric application of loads.

In practice, it is sometimes considered that knots are not as objectionable in columns as in beams, but this opinion is not confirmed by full-size tests. The presence of a knot involves curved grain around it, and to some extent the combined effect of compression on the side of the fibers and tension across the fibers. The strength of wood in both of these respects is relatively very low; hence large knots or smaller knots in groups should be regarded as a sufficient cause for the rejection of timber for columns. Obliqueness of grain also materially reduces the strength of columns and should be avoided. The inspection of timber is discussed further in Art. 83.

The largest single sticks which have been used as columns in this country were 96 ft. high and 2 ft. in diameter from the middle to the bottom. The load to be supported by each stick was over 90,000 lb. They were of Douglas fir from the state of Washington and were employed to raise the granite column of the Cathedral of St. John the Divine in New York.

Figure 53*b* shows an intermediate joint of a trestle post which differs from the ordinary practice in trestle construction, and secures a direct bearing between the ends of the fibers, instead of interposing an intermediate cap between them, which limits the strength to that of bearing on the side of the fibers. The posts and batter posts are without mortise, tenon, or notch. The intermediate cap is replaced by two lighter timbers known as a double or split cap, which are notched over the post; the usual sway braces and the longitudinal struts and girts and X-braces hold the joint in position. In Fig. 53*a* the post is notched as well as the transverse timbers, reducing the section of the post about one-third, but still leaving sufficient bearing area to resist the stress.

In the report of a Committee of American Railway Superintendents of Bridges and Buildings is contained the following statement: " One good method of obviating many of the defects in the mortise-and-tenon joints is by the use of double caps, posts, and sills. That is, instead of using 12- by 12-in. timbers, two timbers 6 by 12 in. can be used, and, being properly fitted and securely bolted together, not only give good results as to strength and durability but expose all defects frequently found in the center of large timbers. Also, being of only one-half the size and weight, they can be handled much more rapidly and consequently much more cheaply and render better access for renewals. Especially is this the case where trains are to be carried during renewals. This method of constructing timber trestles affords greater economy in renewals and has all the advantages of the ordinary mortise-and-tenon construction."



Figs. 53*a* and *b*.—Intermediate Joints of Trestle Posts.

This construction may be still further improved by placing the joint above the double cap. By placing plain fish plates on all four sides, a stronger and more rigid joint can be made than that shown in Figs. 53*a* and *b*. Where still greater strength and stiffness is required, as in splicing the boom of a derrick, a very effective joint may be made by chamfering the edges so as to fit accurately the faces of steel channel fish plates, which are then bolted securely in place. Such a joint is economical and quickly framed.

Figure 53*c* illustrates the general type of laced or trussed column which has been extensively used in exposition buildings, where very high columns are often required. The main section consists of four heavy timbers of square cross-section, say, 12 by 12 in., set in the four corners of a square, with a clearance between them nearly or quite equal to the thickness of each stick. All four sides are then laced with 2- by 12-in. plank inclined to approximately 45 deg., except at the connections with stiffening trusses, which are placed at the height of different stories and connect a series of posts.

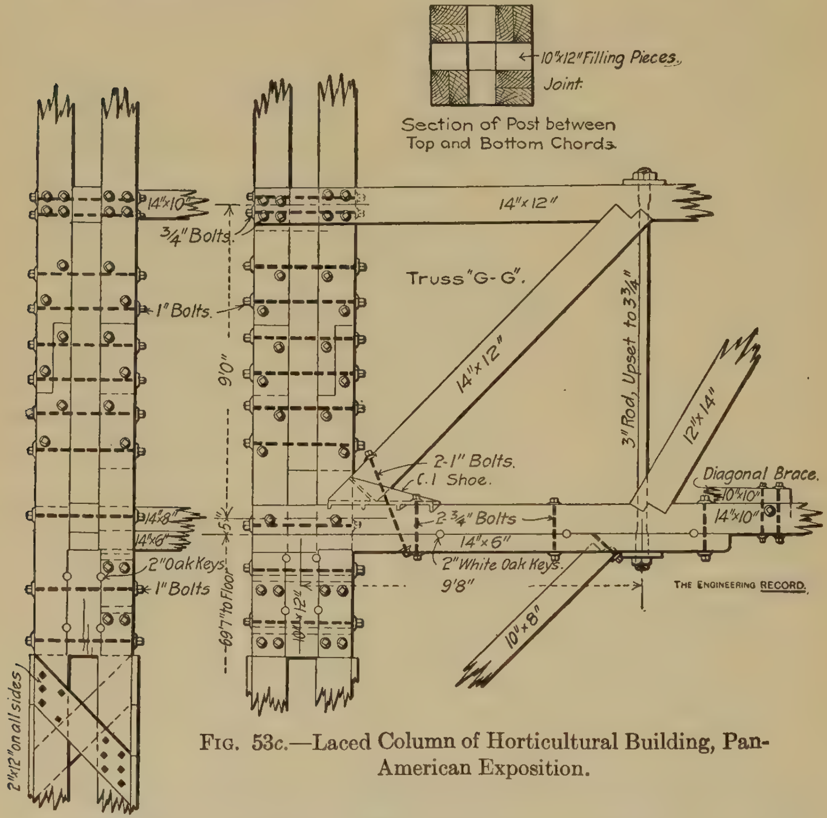
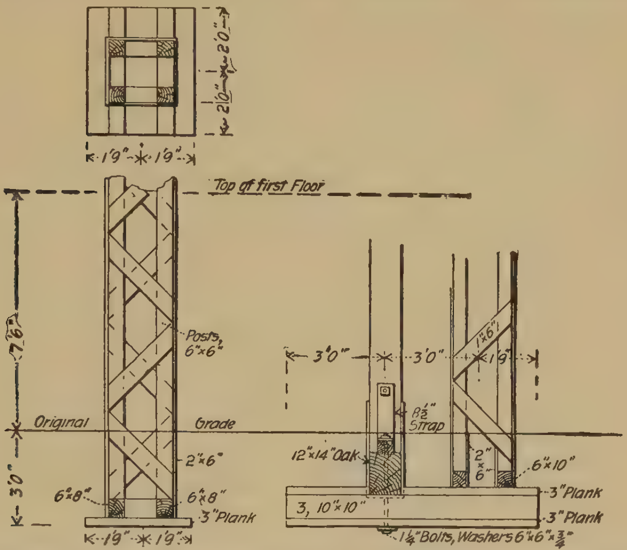

10"x12" Filling Pieces.
Joint
Section of Post between Top and Bottom Chords.
14"x10"
3/4" Bolts.
14"x12"
1" Bolts.
Truss "G-G".
9'0"
14"x12"
3" Rod, Upset to 3 3/4"
12"x14"
2-1" Bolts.
C.I Shoe.
Diagonal Brace.
10"x10"
14"x10"
14"x8"
5 1/2"
2 3/4" Bolts
14"x6"
14"x6"
2" Oak Keys
2" White Oak Keys.
1" Bolts
9'8"
69'7" to Floor
10 1/4"
12"
10"x8"
2"x12" on all sides
THE ENGINEERING RECORD.

FIG. 53c.—Laced Column of Horticultural Building, Pan-American Exposition.

Where trusses, girders, or struts are supported from the sides of the timbers, seats are provided for them on the upper ends of short vertical planks bolted to the face of the column and keyed to it by 2-in. round oak pins driven into holes which are bored half in the plank and half in the column. The drawing also shows the standard construction and splicing of a main column with half-lap joints and with fillers between the timbers, all securely bolted together. It also shows the connection of a stiffening truss which is supported between the heavy timbers composing the column, and indicates the manner in which most of the truss diagonals are stepped into the chord timbers. A sectional elevation showing the general arrangement of columns and stiffening trusses may be seen in Engineering Record, vol. 44, page 182. August 24, 1901.


2'0"
2'0"
1'9"
1'9"
Top of first Floor
7'6"
Posts, 6"×6"
Original
Grade
3'0"
2"×6"
6"×8"
6"×8"
3"Plank
1'9"
1'9"
3'0"
3'0"
1'9"
1"×6"
8½" Strap
12"×14"Oak
2"×6"
6"×10"
3"Plank
3, 10"×10"
3"Plank
1¼"Bolts, Washers 6"×6"×¾"

FIGS. 53*d* and *e*.—Laced Columns and Column Footings, Machinery Building, Columbian Exposition.

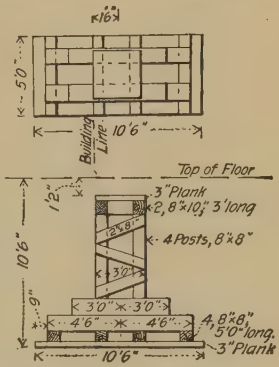
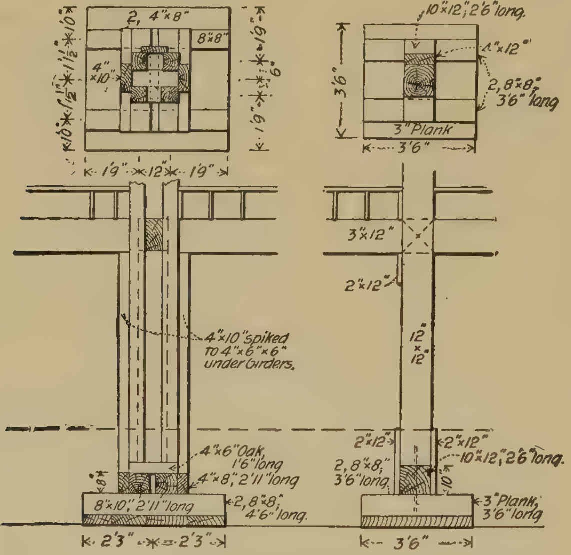

16"
5'0"
Building Line
10'6"
Top of Floor
12"
3"Plank
2, 8"×10," 3'long
12"×8'1"
4 Posts, 8"×8"
10'6"
3'0"
9"
3'0"
3'0"
4'6"
4'6"
4, 8"×8," 5'0" long.
3"Plank
10'6"

FIG. 53*f*.—Column Footing.

Figures 53*d*, *e*, and *f* represent laced columns used in the Machinery Building of the World's Columbian Exposition in 1893. The lacing plank was spiked to the four sticks composing the column. By placing the timbers in the four corners of a square and effectively lacing them, a column may be economically built with a large radius of gyration. Figure 53*g* shows a column in which the four corner sticks are united by vertical planks spiked to them on three sides. The two 4- by 10-in. planks are also employed to take some beam load directly. Figures 53*d*–*i* also show various typical forms of footings designed to distribute the column loads on the sandy bottom, with clay substratum, of the exposition site.

When two sticks only are laced together, they are placed with their diagonals in line and spaced far enough apart so that the lacing on both sides can pass between them. In heavy construction the lacing plank are bolted to the sticks.


2, 4"×8"
8"×8"
4"×10"
10" 1½" 1½" 10"
1'9" 6" 1'9"
1'9" 12" 1'9"
4"×10" spiked to 4"×6"×6" under Girders.
4"×6" Oak, 1'6" long
4"×8", 2'11" long
8"
8"×10", 2'11" long
2, 8"×8", 4'6" long
2'3" 2'3"
10"×12", 2'6" long.
4"×12"
2, 8"×8", 3'6" long
3'6"
3" Plank
3'6"
3"×12"
2"×12"
12"×12"
2"×12"
2, 8"×8", 3'6" long
2"×12"
10"×12", 2'6" long.
10"
3" Plank, 3'6" long
3'6"

FIGS. 53*g*–*h*.—Column Footings, Columbian Exposition.

Composite columns are those in which two or more sticks are bolted or keyed together. These are sometimes used because of the difficulty of getting sticks of large sizes. It is also argued that the average quality of timber in a composite column is higher than that of a solid stick. However, tests indicate that it is practically impossible to make a built-up column act as a single stick.

In Tests of Metals, 1882, page 239, are recorded the results of tests of composite columns composed of two or three sticks of white or yellow pine. In no case did they show materially greater resistance than if each stick were acting freely. The fastenings failed to prevent the slight initial lateral deflection. The lesson drawn from these tests was that in a composite column each element must be designed as if standing alone.

In 1915 H. D. Dewell tested some small composite columns of Douglas fir which were divided into two classes, the first class composed of a number of boards laid face to face and nailed together, and the second class composed of the same type of section as the first class but with the edges tied together with cover plates. It was found that the composite column, 23 in. long, built of four $2\frac{1}{2}$- by $\frac{3}{4}$-in. pieces, with two 3- by $\frac{3}{4}$-in. cover pieces, was as strong as the solid stick, although at the ultimate load the individual pieces separated slightly. Composite beams formed of six 4- by $\frac{1}{2}$-in. pieces proved to have 76 percent of the strength of the solid stick, whereas those composed of four 4- by $\frac{3}{4}$-in. pieces were nearly as strong as the solid stick. The spiking of these composite columns was much more thorough than would be practicable in actual construction.

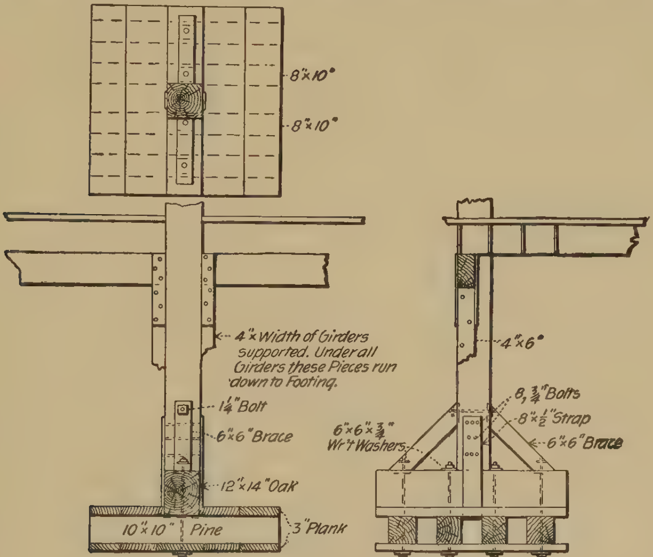

8"×10"
8"×10"
4"× Width of Girders supported. Under all Girders these Pieces run down to Footing.
1¼" Bolt
6"×6" Brace
12"×14" Oak
10"×10" Pine
3" Plank
6"×6"×¾" Wr't Washers
4"×6"
8, ¾" Bolts
8"×½" Strap
6"×6" Brace

FIG. 53*i*.—Braced Column Footings.

As a result of the above tests, and until further tests are available, Mr. Dewell recommends[1] that for the ordinary composite beam the strength shall be taken at 80 percent of the mean of the strengths computed (1) as a solid stick and (2) as a summation of the strengths of the individual sticks considered as individual columns. For composite columns with cover pieces he recommends that the strength be taken as 80 percent of a solid stick of equal cross-section and length.

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[1] See Timber Framing, page 146.

ART. 54. BOLSTERS

A bolster is a horizontal cap piece upon the top of a post to lengthen the bearing of a beam supported by it (Fig. 42*d*). The bearing of the beam is on the side of its fibers, whereas that of the post is on the ends of its fibers. When the beam consists of soft wood, a bolster of hard wood is inserted on top of the post. In Fig. 54*a* large washers or separators are shown between the bolster and beams, apparently to secure ventilation and the drying out of moisture, but this is done at the serious sacrifice of bearing area.

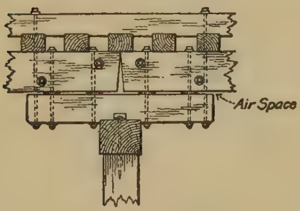


FIG. 54*a*.—Bolster.

When bolsters are used, the effective span assumed in computing the strength of stringers in trestles is sometimes taken shorter than the distance center to center of bents, but this is not warranted by actual conditions. Since the stringers generally decay first and most rapidly at their ends, the effective span then becomes less, but the presence of the bolster permits the beam to be kept in service longer than without it. To reduce the strength of the beam, however, at the beginning simply shortens its period of usefulness.

Besides affording better bearing to the stringers at trestle bents, it aids in uniting the ends of the abutting stringers, and materially stiffens the joint, resisting the churning action due to the repeated application and removal of the live load. In case a bent is washed out, the bolster which is bolted to each stringer acts like a fish plate to assist the splice plate in holding the track from collapsing. Experience has shown that accidents have been prevented by this means.

The objections are frequently made to bolsters that they increase the cost of labor, lumber, and iron, and that additional places are afforded for decay to begin. It is also claimed that the bearing area is not actually increased, since the deflecting stringer brings the load on the end of the bolster. The last objection may be met by sloping the surface of the bolster slightly toward the ends, if this is deemed necessary.

The importance of providing adequate bearing surface for posts and of avoiding the use of soft wood for supports which receive heavy concentrated loads on the side of the fibers is shown in an instructive account of the Collapse of the Floors of a Warehouse in Cincinnati, in Engineering Record, vol. 49, page 490. A number of illustrations of the use of bolsters are given in Art. 56.

The term "corbel" is frequently used interchangeably with bolster in railroad construction, but this use is objectionable, since the designation corbel has for centuries been applied to a different detail of construction in architecture and building, namely, "One of a series of brackets, often ornamental, projecting from the face, especially the external face of a wall; used for support, as of a cornice or string course."

When the end of a post is connected to a beam, cap, or sill, by means of short pieces of plank which are spiked and bolted to both members, the joint is known as a plaster joint (Fig. 54*b*). It is used occasionally in trestle construction, but more frequently in temporary framing of various kinds, especially in falseworks. For a typical illustration of its use in falsework for the erection of a bridge, see Engineering Record, vol. 50, page 271.

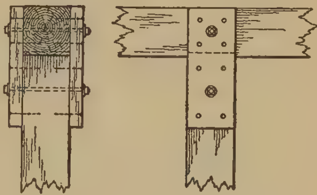


FIG. 54*b*.—Plaster Joint.

Eleven tests of columns with bolsters were made in 1894 in the engineering laboratory of Massachusetts Institute of Technology. There were 8 spruce and 3 oak columns; and 1 yellow pine, 2 maple, and 8 oak bolsters. In nearly every case the column deflected in a diagonal direction and split at the end next to the bolster. The average resistance of the bolster was about 87 percent of the average ultimate strength of the columns. The minimum ultimate resistance was 66 percent of the strength of the column. On the average the spruce columns with bolsters developed 73.4 percent of the strength of the columns without bolsters.

Art. 55. Post Caps

In buildings having several stories with plastered partitions and ceilings, the wooden bolster is objectionable on account of the transverse shrinkage of wood due to seasoning, which causes the settlement of floors and cracks the plaster. Uneven resistance in the bolster also reduces the strength of the column by causing eccentricity of loading. The natural remedy was to substitute cast-iron bearing plates or caps. The settlement of floors, however, was chiefly due to the practice of setting the posts of one story on top of the beams which were laid on the post of the next story below. This arrangement was continued even after cast-iron bearing plates were introduced. The next step in advance was to set the post of the next story above directly on the bolster, or the metal plate or cap, and to support the beams on the projecting ends of the latter. This required the caps to resist bending, although the beams had a narrow bearing directly above the lower column, on account of the smaller size of the upper column. The addition of vertical sides afforded the needed flexural strength for the caps.

The introduction of the cast-iron pintle marked the next improvement, and in a modified form is still in extensive use in slow-burning or mill construction (Figs. 75*d* and 75*j*). The pintle transmits the load from one column directly to the next lower one, and, having a comparatively small horizontal section above its base plate, allows the necessary bearing for the adjacent beams. In all the preceding methods the beams were bolted to bolsters or caps, or fastened together by horizontal straps and other devices, which in case of fire led to serious damage, by pulling down the columns and the floors above. This difficulty was overcome by casting a lug near each end on the bearing plate, which engages a notch in the bottom of the beam, and thus serves to tie the beams together longitudinally, and in case of fire releasing them without injury to the posts. One form of such a cap, which is still in use, is illustrated in Fig. 44*b*.

To perfect the cast-iron post caps it was necessary to extend the sides both up and down so as to bolt them to the top and bottom columns. This arrangement secures a continuous vertical line of posts with sufficient lateral strength to avoid being readily displaced. The ends of the posts bear directly on the horizontal plate of the cap. These features, together with those noted in the preceding paragraph, are all embodied in the Goetz post cap (Fig. 55*a*) which was introduced about 1884, and marked a decided advance in building construction that was promptly recognized by Associations of Fire Underwriters.

Other forms of patented post caps in pressed steel followed later, as the demand for better construction increased. The duplex steel post cap (Fig. 55*b*) was placed on the market in 1902, and the malleable-iron cap in 1907. The Goetz steel post cap is illustrated in Fig. 55*c*. Other forms are known as the Van Dorn and national post caps.

Figure 55*d* shows a steel cap and bracket which was designed in 1897, and since then has been used in some mill building construction. It is intended especially for the connection of beams to wooden columns which are two stories or more in height. It involves very little cutting of the timber and provides for the release of falling beams in case of fire so as not to endanger the stability of the column and other parts of the framework supported by it. Narrow seats, with areas proportioned to the loads on the beams, are cut in the sides of the columns to fit the beveled ends of the wooden floor beams. Horizontal shelf plates are set on them which project far enough to provide sufficient bearing area on the side of the fibers of the beam, and the ends of which are bent down at right angles and bolted to the vertical side or bracket plates, which in turn are fastened to the post by lag screws as shown. The outside bolt on each side of the column passes clear across through both side plates and the vertical ends of the shelf plate, and supports on its side the horizontal plate. The beam is held

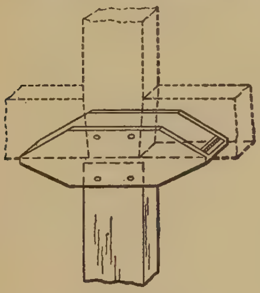


Fig. 55*a*.—Goetz Post Cap.

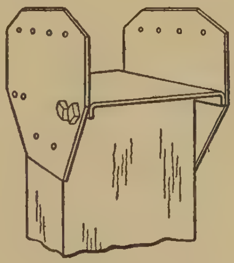


Fig. 55*b*.—Duplex Post Cap.

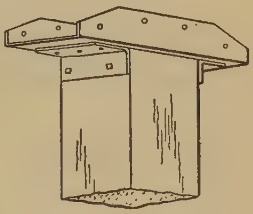


Fig. 55*c*.—Goetz Steel Post Cap.

in place by a single lag screw through each side plate, which is expected to be torn out without injury to the post in case the beam burns through and falls. This post cap was described in Engineering Record, vol. 43, page 307. See also Beam and Column Details in a Factory Building, in Engineering Record, vol. 43, page 526.

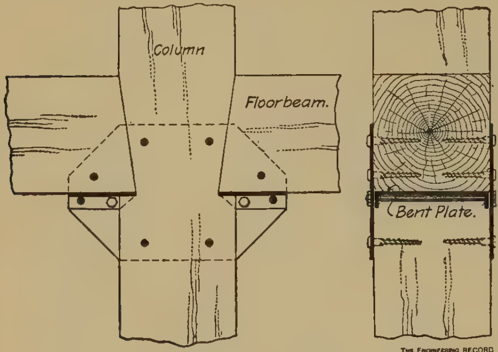

Column
Floorbeam.
Bent Plate.
THE ENGINEERING RECORD

Fig. 55*d*.—Beam Seat on Continous Column.

Figure 55*e* indicates the post and beam connections in a building several stories above that given in Fig. 44*b*. The lowest column is provided with a suitable base plate, which not only affords a square bearing but also protects the timber from the water used in cleaning the floor, and which would otherwise start decay in the joint. For more examples of caps see Art. 75.

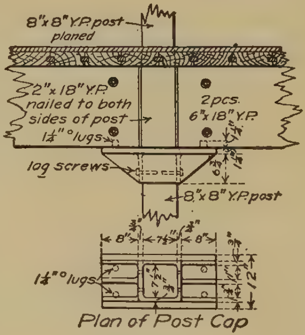
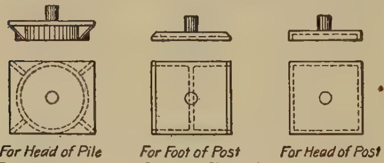

8"x8" Y.P. post planed
2"x18" Y.P. nailed to both sides of post
1¼" ° lugs
2 pcs. 6"x18" Y.P.
log screws
8"x8" Y.P. post
8"
7½"
8"
¾"
12
1¼" ° lugs
Plan of Post Cap

Fig. 55*e*.

Figure 50*f* shows the design of a freight car in which ample provision is made for the bearings against the side of the fibers of the top and bottom chords of the side trusses, for both the diagonals and the verticals. Cast-iron bearing caps are sometimes used in wooden trestle construction, as indicated in Figs. 55*f*, *g*, and *h*. A dowel is cast on the cap to hold the upper timber in position. The caps increase the life of the structure by protecting the joints from holding water.


For Head of Pile
For Foot of Post
For Head of Post

Figs. 55*f*, *g*, and *h*.—Cast-iron Bearing Caps.

Fire tests of columns under load show that the full fire resistance of timber posts is not developed with cast-iron or steel-plate caps, failure being due to excessive heating of the wood at the bearing under the metal cap with resultant crushing and brooming of the fibers and ultimate cracking and slipping of the caps. Tests made at the Underwriters' Laboratories in 1917 and 1918 showed that with metal caps the fire resistance period of timber columns was two and one-half to three times that of unprotected steel and slightly greater than that of unprotected cast-iron columns. By using a concrete cap, Fig. 55*i*, the full fire resistance of the column was developed, doubling the length of fire resistance time. See Engineering News-Record, vol. 87, page 972, December 15, 1921.

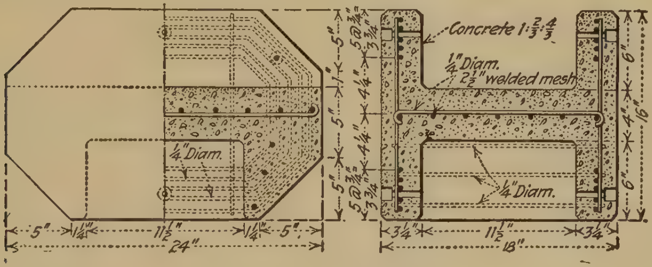
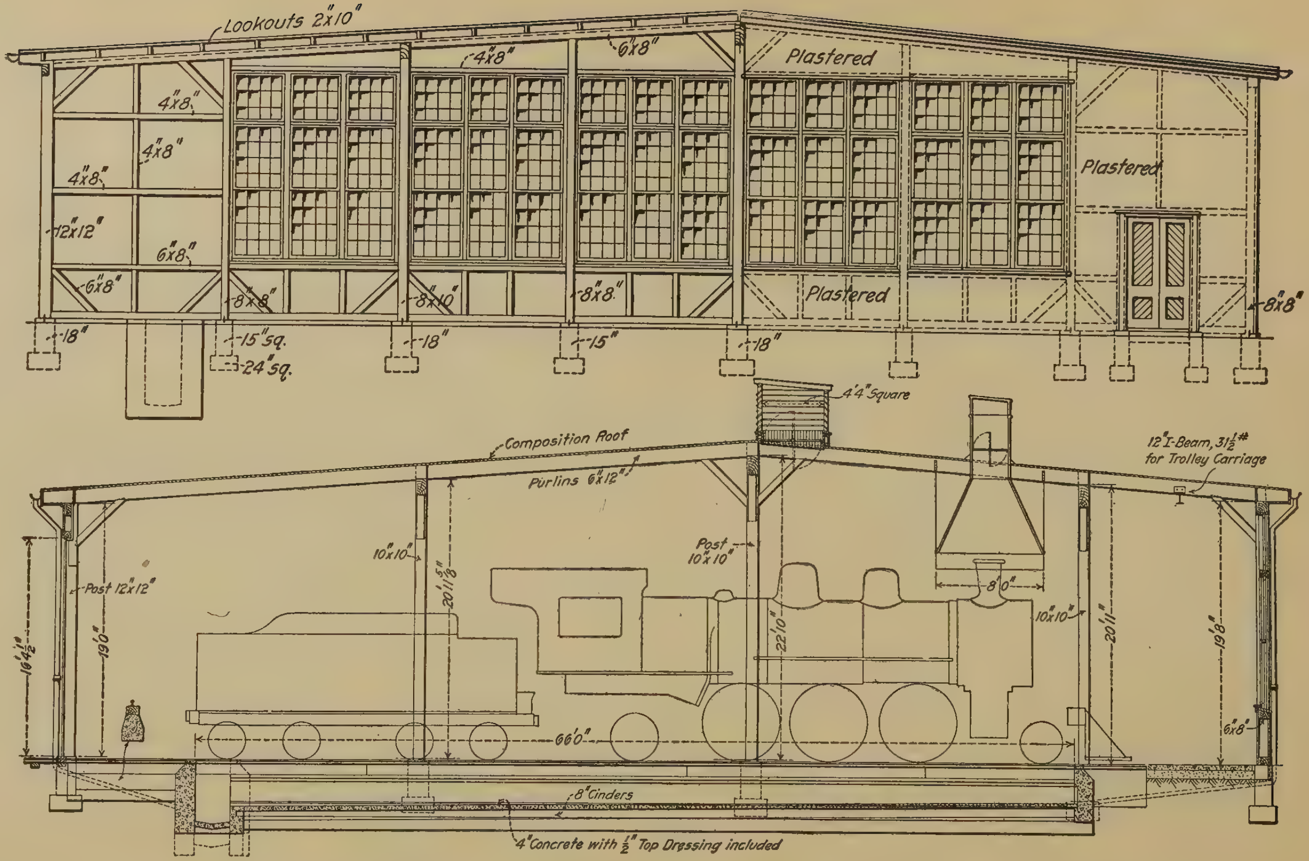


FIG. 55*i*.—Concrete Cap.

ART. 56. ANGLE BRACES

An angle brace is a diagonal strut or tie connecting the vertical and horizontal members of a frame, in order to prevent any change in the angle between them, beyond a small elastic deformation. Beams are frequently connected to their supporting columns by an angle brace on each side of the column. Illustrations of their use are given in Arts. 70 (Plates III and IV), 73 and 81, as well as in this article.

The principal function of braces is to resist compression, and for this purpose the toe at each end engages a notch in the adjacent member, forming a step joint (Fig. 56*e*). Frequently, however, a mortise-and-tenon is added, and a treenail connects the two pieces to hold them rigidly together. A number of excellent illustrations of the use of angle braces are given on the inset accompanying an article on the Intramural Elevated Electric Railway of the World's Columbian Exposition, in Engineering News, vol. 28, page 362, October 20, 1892. The braces at the feet of the posts are bolted to the posts and sills. Cast-iron braces and brackets

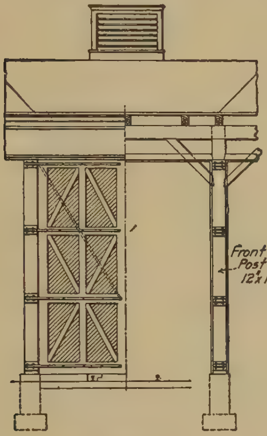

Lookouts 2"x10"
4"x8"
6"x8"
Plastered
4"x8"
4"x8"
4"x8"
12"x12"
6"x8"
6"x8"
8"x8"
8"x10"
8"x8"
Plastered
Plastered
8"x8"
18"
15" sq.
24" sq.
18"
15"
18"
4'4" Square
Composition Roof
Purlins 6"x12"
12" I-Beam, 31½#
for Trolley Carriage
Post 12"x12"
16'4½"
19'0"
10"x10"
20'11⅛"
Post 10"x10"
22'10"
8'0"
10"x10"
20'11"
19'8"
6"x8"
66'0"
8" Cinders
4" Concrete with ½" Top Dressing included

R. R. Age Gaz.

FIGS. 56*a* and *b*.—Elevation of End Walls and Cross-section of Round House at Decatur, Ill., Wabash Railroad.

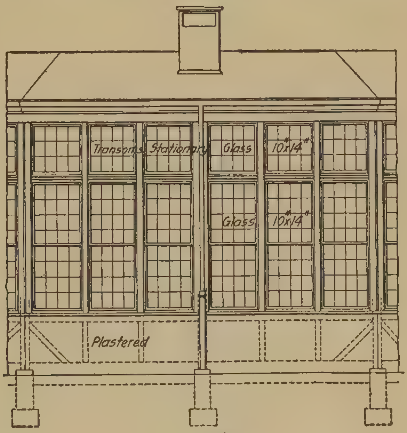
are used in some cases to stiffen the top of the bent. In Fig. 56*c* the angle braces are framed into bolsters, which are keyed to the sloping beams supporting the roof.

Figure 56*a* shows the framing of the end wall of a round house, in which an unusually large percentage of the area is glazed. The balance is covered with a heavy coating of cement plaster on expanded metal, which forms the finish of the wall both inside and out. The timber frame is braced by means of short diagonal braces, as indicated on the diagram, the left side of which shows the framing and the right side the finish. The posts are held in position on the concrete footings by dowels. In Fig. 56*b* the braces connecting the posts to the roof timbers, both longitudinally and transversely, may be seen. The inner wall has solid doors with diagonal battens (see Fig. 56*c*), and the short space between them and the roof is likewise finished with plaster. For the description of other features of the building, see Railroad Age Gazette, vol. 45, page 1298, and Journal Association of Engineering Societies, vol. 41, page 277.



Front Elevation.

FIG. 56*c*.



Rear Elevation.

FIG. 56*d*.

A series of comparative tests of bracing for wooden bents was made in 1897 at the Michigan College of Mines, the results of which were presented to the Lake Superior Mining Institute, in a paper by Edgar Kidwell, and published in vol. 4 of the Proceedings. Two forms of wooden angle braces were used, one with a

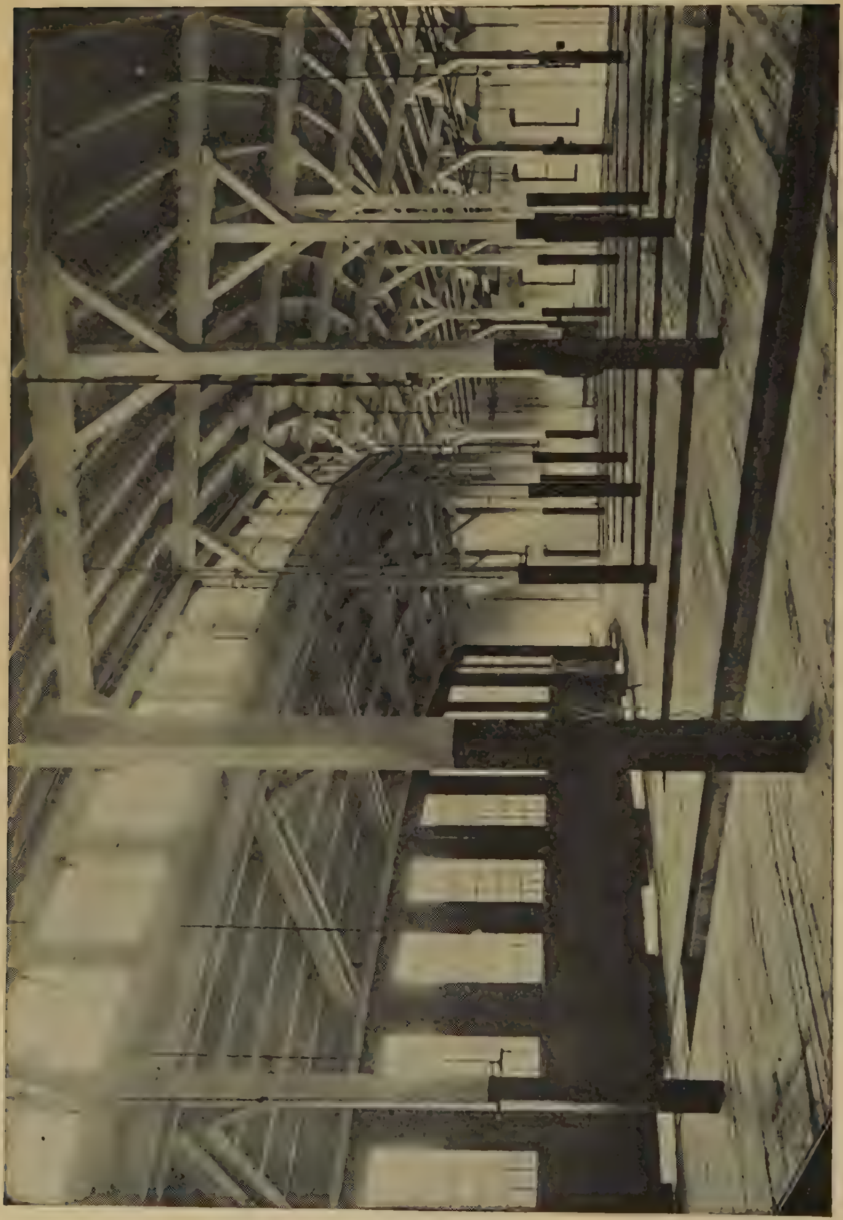


FIG. 56e.—Round House of Ontario & Western Railroad at Middletown, N. Y.

single step joint accurately framed to a driving fit, the brace being fastened with wire nails; and the other without the step, but connected with mortise, tenon, and treenail. Another bent had full diagonal braces; three others had cast-iron brackets or knees with different details, such as are in use in the mining regions; and cast-iron braces of T-section, with bearing plates and lugs at the ends, were used with two methods of fastening.

The tests were made for both strength and stiffness. The results indicate that " when new and under conditions in average practice, the wooden braces are fully equal in rigidity and strength to any form of iron braces now in use. The strength of a structure in which wooden braces are used will not be much affected when the braces shrink, since a slight distortion of the building will bring them unto play again; but the shrinkage will certainly destroy the rigidity of the structure, and it is this fact which has caused some of the mines of Lake Superior to abandon the wooden braces for iron ones. . . . The experiments seem to indicate that, for deflections within a reasonable limit, the bents with wooden braces are the better, and, if restrained or loaded on top, they will exceed in rigidity, and probably in strength, the bents braced with iron knees of the usual pattern. . . . No form of iron knees tested gave a very rigid bent, as the latter had to be deflected more than could be allowed in practice before the bents exhibited much strength. If iron knees are used, it is better to use the plain pattern, instead of the usual form provided with lugs or cogs. The cast-iron brace gives a more rigid structure than any of the iron knees, and when merely spiked on is strong enough for any structure not subject to heavy shocks." Kidwell's paper gives full details of loads and deflections, half-tone views of the bents upon completion of the tests, and diagrams showing the relative deflections of the bents.

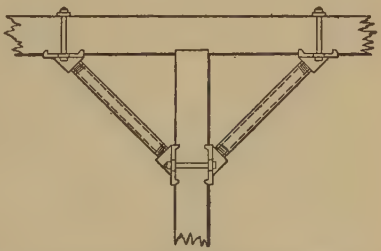


Fig. 56*f*.—Kidwell Angle Brace.

Figure 56*f* shows an adjustable type of angle brace, consisting of cast-iron angle blocks, with lugs to engage notches in post and beam, and a pipe with right and left threads at the ends to screw into the angle blocks.

CHAPTER IV

WOODEN ROOF TRUSSES

Art. 57. Types of Roof Trusses

A truss consists of a frame in which each member is subject only to tension or to compression. To avoid flexure in the members their axes should intersect in a point at each joint, and the loads should be applied only at the joints, which are called panel points. Geometrically, a truss is composed of a series of triangles. Theoretically, the members of a truss are free to turn at the joints, but practically the joints are more or less rigid, due to the methods of construction employed.

The upper line of members from one support to the other forms the upper chord, and the lower line of members the lower chord. The members connecting the chords are called web members or braces, some of which take compression and are known as struts, while others take tension and are called ties. A simple truss is one having only two supports, and under vertical loads has vertical reactions.

A roof truss is designed especially to support a roof, but it may also support a ceiling, and, in special cases, may support one or more floors below by means of rods, when it is necessary to avoid all columns in any given story. In ordinary construction the roof covering and the laths or sheathing underneath it are supported by rafters which are parallel to the slope of the roof surface; the rafters rest on purlins, which are horizontal beams extending from one truss to another at the panel points of the latter; and the trusses are supported on the wall plates of the outer walls of the building. In special cases one or more of these elements may be omitted or replaced, as, for example, when the purlins are spaced close together and support the roof covering directly without the aid of rafters, or when a heavy sheathing of planks is used to support the covering of slate or tile, and rests directly on the trusses. When the purlins rest on the upper chord of a truss between panel points, the chord members are subject to flexure or bending in addition to compression, and the truss then fails to conform to the definition of a true truss.

A wooden roof truss is built principally of wood, but may have iron or steel rods for its tension web members. When the lower chord of a truss is also constructed of steel, it is generally known as a combination truss. Various kinds of fastenings are used at the joints; the timbers composing the chords usually continue past the intermediate joints and are spliced between panel points; while the web struts are connected to the chords by many of the forms of joints or fastenings described in Chapters I and II. In wooden trusses the tension rods pass through the chord timbers and take bearing on washers, while in combination trusses pins are used to connect the members meeting at those joints, respectively, where two or more iron or steel members come together. See illustrations in Arts. 70, 72, and 73.

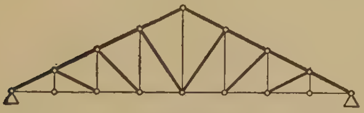


Fig. 57*a*.—English Roof Truss.

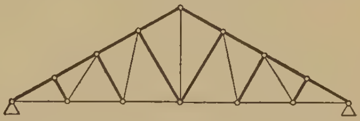


Fig. 57*b*.—Belgian Roof Truss.

Figures 57*a* to *h* give the skeleton diagrams of the principal types of wooden roof trusses. In Fig. 57*a* the truss has vertical tension rods and diagonal wooden struts. This type is more extensively used than any other. The number of panels depends upon the span, and when there are only two panels it becomes the king-post truss. When the king-post truss was built entirely in wood, the vertical tension member was called the king-post, but at present a rod is nearly always used instead.

In Fig. 57*b* the wooden struts are perpendicular to the upper chord and the diagonal tie-rods slope upward toward the center. The struts are shorter than in the preceding form, provided the rise is the same. The number of panels in the lower chord is two less than in the upper chord. This type is not adopted as frequently as its merits deserve. When the four middle panels are omitted, this form reduces to that of the combination truss in Fig. 73*c*.

When it is desired to construct the Belgian truss with the longest struts both meeting the lower end of the vertical center rod, it is necessary to have the rise of the truss conform to this requirement. For a truss of six panels the rise must be $l/\sqrt{8}$ or 0.3536l, in which l is the effective span. For eight panels the rise is $l/\sqrt{12}$ or 0.28867l; and for ten panels, the rise is $l/\sqrt{16}$ or 0.25l. There is frequently no objection, however, to using a lower and more economical rise than that of 0.3536l, by using two inclined rods diverging from the peak of the truss.

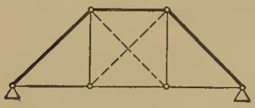


FIG. 57*c*.

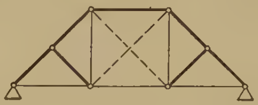


FIG. 57*d*.

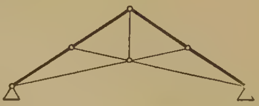


FIG. 57*e*.

Figures 57*c*, *d*, and *e* give forms suitable for very short spans, the first two requiring counter braces in the middle panel as indicated by broken lines, to resist wind pressure. Figure 57*e* shows what is called a scissors truss. The prolongation of the lower chord members past the center should be indicated as compression members in the diagram. This form is often used for roofs having a steep slope.

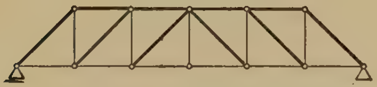


FIG. 57*f*.—Howe Truss without Counters.

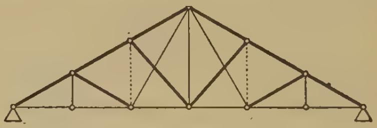


FIG. 57*g*.

The truss in Fig. 57*g* is well adapted for construction in wood throughout. The long diagonal tension members serve to stay the longer struts by being connected at their intersection; but such connection does not interfere with the stresses in either. The dotted lines do not represent members, but merely the fact that the two panel points are in the same vertical.

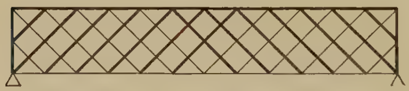


FIG. 57*h*.—Town Lattice Truss.

Figures 57*f* and *h* illustrate trusses with horizontal chords; these are used either to support flat roofs or to serve the purpose of trussed purlins for a sloping roof. Both of these types are also used for bridge trusses, but the former then requires counter braces on account of the moving load. The details of the Howe truss are very simple. The lattice truss is built in wood throughout and all of its members may be composed of planks which are connected at the intersections with wooden pins, and occasional bolts to keep the planks in contact.

Sometimes the truss of Fig. 57*f* is modified slightly by inclining the upper chord parallel to the slope of a roof as in Fig. 72*a*. The upper truss on Plate V has an inclined upper chord, but the web members form a series of triangles with varying heights, their corresponding sides toward the middle being parallel to each other. A similar form is given in Fig. 73*b*. A modification of the form of Fig. 73*c*, which is adapted to six panels, is given in the combination truss on Plate V.

For spans of about 30 ft. or less the roof covering may be supported by inclined beams and purlins as on the left side of the freight station shown on Plate V. On the right side the beams are lighter in section, and are reinforced by a straining beam and sloping struts; the purlins are small in section and closely spaced. For spans exceeding 30 ft. it is usually necessary to provide either intermediate supports, as by a line of columns, or to employ trusses.

The economical limits of span between which wooden roof trusses of various types may be used, as determined by the experience of architects and builders, are given in Chapters I and III of Trussed Roofs and Roof Trusses, by the late F. E. Kidder, or in Architects' and Builders' Pocketbook, by the same author and the late Thomas Nolan.

PROBLEM 57. Consult the references in the preceding paragraph and prepare a table containing the economical limits of span for the different types of wooden roof trusses.

ART. 58. WEIGHTS OF ROOF TRUSSES

The loads which are to be supported by a roof truss consist of the dead load, which includes the weight of the truss itself and of the roof covering and other fixed loads supported by it, the snow load, and the wind load. The snow and wind loads are usually specified, or must be determined by the designer in accordance with the climatic conditions of the locality where the building is to be erected. Sometimes a building is in a sheltered location that may permit the wind load to be reduced; but frequently the effect of high winds is not given sufficient consideration in the design of roofs and their supporting framework. The general values employed by engineers are given in Roofs and Bridges, Part I, Art. 2, and in Part II, Arts. 17 and 20.

The most complete list of weights for all the different roof coverings in use in this country, and tables of approximate weights of rafters, purlins, and roof trusses are given in the handbook to which reference was made in the last article.

By comparing the classified weights of 121 designs of roof trusses of spans ranging from 48 to 96 ft., with a rise of one-sixth to one-third of the span, spacing of trusses from 8 to 12 ft., a snow load of 25 lb. per sq. ft., and a normal wind load corresponding to a pressure of 40 lb. per vertical square foot, the following formula was deduced for the weight of wooden roof trusses of the types indicated in Figs. 57*a* and *b*:

$$W = \tfrac{1}{2}al(1 + 0.15l) \qquad (1)$$

in which W is the weight of one truss in pounds, a the distance between centers of trusses, and l the span, both a and l being expressed in feet.

The designs were made in 1908 for longleaf yellow pine and Western hemlock, all sizes of timbers being in full inches, whether odd or even, thereby securing a fairer basis for comparison. It was found that the trusses of the latter wood are about two percent heavier than those of the former, but that is chiefly due to using relatively lower unit stresses than those given in Art. 89. The average weights of the trusses for the two types used, namely, the English and Belgian (Figs. 57*a* and *b*), are practically the same. A rise of one-fourth of the span gives the lightest truss. For spans below 72 ft., the six-panel trusses are lighter than those with eight panels, whereas for a span of 72 ft. the difference is very slight, thus indicating that 12 ft. is about the economical panel length. The weight of cast iron and steel in the trusses varies from 12 to 20 percent of the total weight, the average being 15.3 percent. The percentage diminishes as the span increases. If 10 percent of the lowest and 10 percent of the highest values are omitted, the percentages range from 13 to 18.

Since the entire weight of the members, including the details, cannot be computed with precision until the details are designed and the drawing made, it is desirable to compare the total weight of the truss with what may be termed the theoretic weight, which is obtained by means of the adopted gross section areas of the members and their lengths from center to center of panel points. The percentage to be added to the theoretic weight to obtain the actual weight was found from an analysis of 97 designs to vary from 13 to 40 percent, the average being 22.4 percent. The percentage also diminishes as the span increases, but does not appear to depend upon the ratio of rise to span. If 10 percent both of the lowest and of the highest values are omitted, the percentages range from 14 to 33. The effect of using unit stresses of three-quarters of the magnitude of those for longleaf yellow pine is to increase the average percentage to be added to the theoretic weight from 18.2 percent to 25.6 percent. It must be remembered that the unit stresses for iron and steel remain unchanged, whereas a large proportion of the weight of details to be added to the theoretic weight consists of cast-iron and steel details.

An investigation was made by N. Clifford Ricker for the purpose of obtaining a more accurate formula for the weight of wooden roof trusses than those in existence at the time, the results of which are published in Bulletin 16 of the University of Illinois Engineering Experiment Station. The type of truss chosen is Fig. 57*a*, and the kind of wood is longleaf yellow pine. The general arrangement taken was to have a purlin at each panel point of the truss, which supported rafters on which was laid $\frac{7}{8}$-in. sheathing, covered by painted tin. The snow load was assumed at 20 lb. per horizontal square foot, and a normal wind load corresponding to 30 lb. per vertical square foot.

Ten trusses were designed for spans of 20 to 200 ft., varying by 20 ft., with a rise of one-fourth of the span. The panel length was taken 10 ft. in all cases, and the spacing between trusses 20 ft. The following formula was deduced from the computed weights of these designs:

$$w = \frac{l}{25} + \frac{l^2}{6000} \qquad (2)$$

in which w is the weight of the truss in pounds per square foot of horizontal projection of the roof supported, and l the span in feet. A similar series of designs was made by using white pine instead of longleaf yellow pine, and the trusses were found to be somewhat lighter.

A comparison of five designs of 200-ft. trusses, with a rise of 50 ft., and spaced 10, 15, 20, 25, and 30 ft. respectively, shows that the total weight of the roof is a minimum for the spacing of 15 ft. A comparison of trusses of the same span, and spaced 20 ft., but with 8, 10, 12, 14, 16, 18, and 20 panels, respectively, gives a minimum total weight of roof for panels 20 ft. long. Another series of trusses of the same span and spacing, but with rises of 20, 25, 30, 35, 40, 45, and 50 ft., respectively, indicate that the total weight of roof is a minimum for a rise of 35 ft., which is a little over one-sixth of the span. the angle of inclination being nearly 20 deg.

Still another series of trusses of the same span and spacing was designed, with from one to five purlins for each panel length, subjecting the upper chord to combined flexure and compression. The results show that the weight of the roof covering, sheathing and rafters combined decreases as the number of purlins is increased; the weight of the purlins increases, the additional weight of the connections slightly diminishes, and the weight of the truss increases. Considering the total weight of roof, it was found that no advantage results from the use of more than two purlins per panel 25 ft. long, or of more than one purlin for panels of ordinary length. Additional designs show that it is not economical to raise or camber the lower chord, indicating that this is done only for the sake of appearance.

Problem 58. A roof truss has a span of 80 ft., and the spacing is 14 ft. Compute the weight of the truss by formulas (1) and (2) and find the percentage of difference between the results. Make a similar computation for a span of 100 ft. and a spacing of 12 ft.

Art. 59. Stresses in Rafters

A common rafter which supports a roof covering is preferably regarded as a simple beam under uniform load, the direction of which is such that one component is parallel to the rafter and the other perpendicular to it. In Fig. 59*b*, the vertical arrows represent the weight of roof covering, sheathing, and the rafter itself, as well as of the snow load; and the arrows perpendicular to the rafter represent the wind load. In Fig. 59*c*, the decomposition of the total load into the longitudinal and normal components W_l and W_n is indicated, but it is to be remembered, however, that the loads are distributed, and not concentrated.

In Fig. 59*a*, the parabola *amb* gives the bending moment diagram for the rafter due to the normal load, but it is drawn to such a scale that the ordinates represent the unit stresses in the outer fiber due to flexure. The ordinate at the middle equals $6\ W_n l'/8\ bd^2$, in which l' is the inclined span of the rafter, b and d the breadth and depth of its cross-section.

If the rafter is regarded as fastened to the purlin at the lower end only, then the unit compressive stress due to the longitudinal component of the load varies as the ordinates of the triangle *abc* in which *bc* equals W_l/bd. The maximum unit stress is therefore represented by the maximum ordinate $y = y' + y''$, which is located at the point of contact *t* of a parallel to *ac* drawn tangent to the parabola. By mechanics, the section is located at a distance from the upper support equal to $x = \frac{1}{2}l' + \frac{1}{6}d \tan \alpha$ in which α equals the angle of inclination of the rafter to the horizontal. If, however, the rafter is regarded as fastened only at the upper end, the maximum unit stress in the outer fiber is tension, and the section in which it is located is the same as before. This shows that the maximum stress is so near the middle that practically its value may be taken the same as that at the middle of the span.

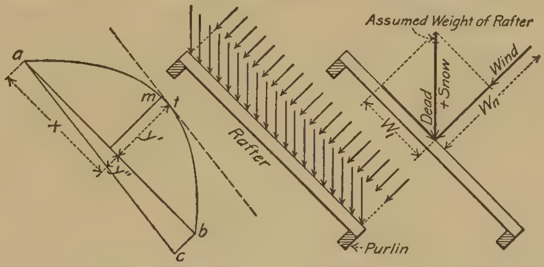


FIG. 59*a*. FIG. 59*b*. FIG. 59*c*.

For example, let l = 10 ft. = 120 in., α = 60 deg., and let the depth of the rafter be assumed as 6 in. Then

$$x = \tfrac{120}{2} + \tfrac{1}{6} \times 6 \times 1.73 = 61.73 \text{ in.},$$

or only 1.73 in. below the middle of the span. The total vertical load given, including slate, sheathing, rafter, and snow, is 562.5

lb., and the normal wind load is 800 lb. The longitudinal component is $562.5 \times 0.866 = 487$ lb., and the total normal component is $800 + 562.5 \times 0.5 = 1081$ lb. The maximum bending moment is $1081 \times 120/8 = 16{,}215$ in.-lb. For an allowable unit stress in the outer fiber of 1800 lb. per sq. in., the resisting moment of the rafter is

$$1800 \times b \times 6^2/6 = 10{,}800b.$$

Equating these moments, there is found $b = 1.50$ in. Taking a width of 2 in., the direct compression [or tension] is found to be $\frac{1}{2}(487)/(2 \times 6) = 20$; and that due to flexure is

$$16{,}215 \times 6/(2 \times 6^2) = 1351;$$

making the total unit stress 1371 lb. per sq. in. These results show that the unit stress due to the longitudinal component of the load is so small in comparison with that due to the normal component that it may be neglected in the design of rafters. A rafter is seldom given as steep a slope as 60 deg., so that generally the direct stress is relatively smaller.

PROBLEM 59*a*. Ascertain whether a 2- by 5-in. rafter may be used in the example given above, without exceeding the allowable unit stress of 1800 lb. per sq. in.

PROBLEM 59*b*. A 2- by 6-in. rafter on a roof truss having a rise of one-fourth of the span is supported by purlins spaced 12.02 ft. center to center. Its total vertical load, including its own weight, is 670 lb., and the normal wind load is 572 lb. Compute the maximum unit stress in the rafter.

ART. 60. STRESSES IN PURLINS

When common rafters are employed, a purlin is practically a simple beam supporting concentrated loads at regular intervals, each of these loads being the resultant of the normal and vertical loads on one rafter, including the weight of the latter. The purlins are frequently placed with their longer sides vertical; and though this direction gives the largest resistance of the purlin to the vertical loads, it may be quite unfavorable for the wind loads, the maximum stress being produced when both vertical and normal loads act together. It is possible for this arrangement of the purlin to render it weaker than if it were placed with its longer sides parallel to any other direction. This condition occurs when the neutral axis coincides with the diagonal of the cross-section.

The purlin will have its largest resisting moment when the longer sides are placed parallel to the direction of the resultant of its maximum loads. When beam hangers are employed to support the purlin from the upper chords of the trusses, this position should be used, for then the cost will not be greater than for any other position.

For ordinary roof trusses in which the rise does not exceed about one-third of the span, it is customary to rest the purlins on the upper chord, thus making the longer sides of any purlin perpendicular to the slope of the rafter, and avoiding the necessity of notching the purlin over the chord of the truss. It may frequently be possible, however, that the cost of the notching is less than the amount of lumber saved in the purlins.

When the purlins rest on the upper chord, they are designed by finding the maximum stress due to each of the two sets of loads, one acting normal to the roof and the other tangent to it, and then adding these stresses.

For example, let it be required to design a purlin having a span of 12 ft. and supporting rafters 18 in. apart, the normal component of the load on each rafter being 828 lb., and the longitudinal component 228 lb.

The maximum bending moment from the normal rafter loads is

$$M' = \tfrac{1}{2}(828 \times 7)6 - 828(1.5 + 3 + 4.5) = 9940 \text{ ft.-lb.}$$

The maximum bending moment from the longitudinal rafter loads is

$$M'' = \tfrac{1}{2}(228 \times 7)6 - 228(1.5 + 3 + 4.5) = 2735 \text{ ft.-lb.}$$

Assuming a section of 8 by 10 in., the weight of the purlin is 214 lb. at 36 lb. per cu. ft. Resolving this weight into two components, the normal component is $214 \times 0.8944 = 191.4$ lb. and the longitudinal component is $214 \times 0.4472 = 95.7$ lb., the angle of inclination of the roof surface being 26 deg. 34 min. The bending moment due to the normal component is $\frac{1}{8} \times 191.4 \times 12 = 287.0$ ft.-lb., and that due to the longitudinal component is $\frac{1}{8} \times 95.7 \times 12 = 144.1$ ft.-lb. Adding these to the above values the total normal moment $M' = 9940 + 287 = 10{,}227$ ft.-lb., and the total longitudinal $M'' = 2735 + 144 = 2879$ ft.-lb. The stress due to M' is $\frac{6 \times 10{,}227 \times 12}{7.50 \times 9.5^2} = 1090$ lb. per sq. in. and that due to M'' is $\frac{6 \times 2879 \times 12}{9.5 \times 7.50^2} = 388$ lb. per sq. in., making the total stress 1478 lb. per sq. in. This design is satisfactory for dense-select Douglas fir, the allowable stress being 1750 lb. per sq. in. (Art. 89).

Art. 61. Design of a Roof Truss

The span and loading of a roof truss to be designed are generally given in the specifications, as well as the working unit stresses to be used for the different kinds of material which are to be employed in its construction. Sometimes also the type of truss preferred may be specified, and conditions imposed which control the spacing, kind of roofing material, etc. For the purpose of encouraging in students the habit of systematic arrangement in computations, of facilitating reference to preceding portions of the design, and of simplifying the examination of the reports, the following order of design is suggested:

ORDER OF DESIGN

1. Rafters. Length of upper chord of truss on one side. Inclined span of rafter, or distance between purlins. Roof area supported by one rafter for this span, in terms of spacing x. Weight of sheathing, roof covering, snow, and estimated weight of rafter for this area. Combined dead and snow load. Angle of inclination of roof surface. Normal wind load per square foot, and for the given area. Total normal components of loads. Longitudinal component. Resisting moment for rafters of 2 by 4, 2 by 5, 2 by 6, 2 by 7, and 2 by 8 in. Corrected weight of rafter.

2. Purlins. Span. Revised normal and longitudinal load transmitted by each rafter. Bending moments from normal and longitudinal loads. Assumed section of purlin. Normal and longitudinal bending moments due to weight of purlin. Total moment in each direction. Maximum unit stress in outer fiber. Revision, if necessary. Final weight of purlin.

3. Estimated Weight of Truss. Weight per panel. Weights

of roof covering, sheathing, snow, and rafters for one panel load determined directly from rafter load. Weight of one purlin. Total dead panel load on upper chord. Snow panel load. Dead panel load on lower chord due to ceiling, if any. Wind panel load.

4. Computation of exact lengths of truss members, center to center of panel points. Diagrams showing these dimensions, all panel loads, combined dead and snow load reactions, and wind load reactions.

5. Truss diagrams drawn to as large a scale as prescribed sheet will allow. See plate posted for general arrangement and notation. Drawing of load line, all panel loads and reactions being laid off in regular order while passing completely around the truss in a clockwise direction. Find dead and snow load stresses by a single diagram, using as large a scale as practicable. Construct wind stress diagram to a larger scale. Mark stresses [expressed in multiples of 10 lb.] on left half of truss diagram in three lines: 1, dead and snow; 2, wind; 3, maximum.

6. Design of sections of truss members. Arrange result in tabular form with the following column headings: Member, length, maximum stress, size ratio l/d, f, section area required, section area furnished, and remarks. Place on the same page a skeleton truss diagram with panel points designated by letters. When upset screw ends are used, give diameter in column of remarks. Leave dimensions of cross-section and section area furnished by lower chord blank until end joint is designed.

7. Design of washers, arranged in regular order for various panel points. For ogee or plate washers: net bearing area; diameter of bolt; diameter of hole; gross area of base; diameter of round, or side of square washer; net area furnished. For beveled washers: longitudinal and normal components of stress in rod; bearing areas of end and base of washer; depth of notch, width, length; long diameter of nut; detail pencil drawing to scale with dimensions, indicating intersection of the three resultant pressures.

8. Other details of intermediate joints arranged in regular order for various panel points. Angle between section and direction of fibers. Allowable unit compression. Bearing area of strut. Depth of notch or indent. Size and location of dowels. Pencil drawing, with dimensions.

9. Design of end joint of truss. Bearing surface and depth of

toe of upper chord. Bearing on horizontal plate. Bearing surface, number, and depth of lugs on forged shoe. Thickness of lugs. Clear distance between lugs, and from outer lug to end of lower chord timber. Required net section of lower chord. Net and gross depth. Diameter of inclined bolt. Size of washer. Design of key between chord and bolster. Bearing area and depth of notch. Length required to resist shear. Length required to resist moment of rotation of key. Size of bolt and washers to resist vertical reactions of key. Clear distance between inner lug and key. Detail pencil drawing to scale, with dimensions.

10. Connection of purlins to upper chord of truss. Number and size of nails to resist component of resultant load parallel to upper chord. Investigate bearing surfaces of purlins. Other details, if any.

11. Design of anchorage. Width of wall plate. Depth of notch over wall plate. Connection to resist lifting of truss by internal wind pressure. Anchor bolts.

12. Design of splices. Splice in upper chord. Splices in lower chord.

13. Detail drawing, with complete dimensions required for construction. See Plates I-IV, blue prints, or other drawings exhibited, showing arrangement, dimensions, lettering, and execution.

14. Bill of material, properly classified, for one complete bay of roof.

15. Estimate of cost for one complete bay of the roof.

16. Analysis of weight of one roof truss. Weight of lumber. Weight of steel. Weight of cast-iron. Weight of metal expressed as percentage of total weight. Theoretic weight. Ratio of computed or final weight to theoretic weight. Excess or deficiency of estimated weight of truss with respect to final weight.

ART. 62. SPECIFICATIONS, RAFTERS, AND PURLINS

SPECIFICATIONS

Let it be required to design a wooden roof truss of the type illustrated in Fig. 57*a*, with a span of 60 ft., a rise of one-fourth of the span, and six panels, the spacing between centers of trusses being 12 ft. The truss is to be designed for construction of

Douglas fir with vertical tie-rods of structural steel, plates of steel, plate washers of steel and ogee washers of cast iron. The roof covering is to consist of slates, $\frac{3}{16}$ in. thick, laid on 1-in. sheathing, supported by rafters spaced not less than 16 in. nor more than 24 in. apart, the rafters being carried by purlins placed at, or near to, the panel points of the trusses. The sheathing, rafters and purlins are also to be of Douglas fir, which is to be assumed to weigh 36 lb. per cu. ft.

The snow load is to be 20 lb. per square foot of horizontal projection and the normal wind load is to be computed from Duchemin's formula for a pressure of 30 lb. per vertical square foot. This formula is

$$P_n = P\frac{2 \sin \alpha}{1 + \sin^2 \alpha}$$

in which P_n = intensity of normal pressure upon the given surface, P = intensity of normal pressure upon a vertical surface, and α = angle made by the surface with the horizontal. In proportioning different parts of the structure provision is to be made for the maximum stresses due to dead, snow, and wind loads.

The allowable unit stresses expressed in pounds per square inch are to be:

For dense-select Douglas fir (see Art. 89): Extreme fiber stress in bending, 1750; modulus of elasticity, 1,600,000; shearing parallel to the fiber, 210; longitudinal shear in beams, 105; compression perpendicular to the fiber, 380; compression parallel to the fiber, 1713; compression in columns under 10 diameters in length, 1285; and for columns of greater length, the values given in Art. 90.

For steel tie-rods and bolts in tension, 18,000; for steel in flexure, 18,000; for rivets in shear, 12,000; for rivets in bearing, 24,000.

DESIGN OF RAFTERS

To obtain the span of the rafters it is necessary to compute the length of the upper chord for one-half of the truss span. This is most conveniently done with the aid of a table of squares of the modern type. The half-span of the truss is 30 ft. and the rise is 15 ft.; the length of the upper chord is 33 ft. 6½ in., and hence the inclined span of a rafter is 11 ft. $2\frac{3}{16}$ in., or 11.18 ft. The horizontal projection of its span is 10 ft.

Let x be the distance in feet between centers of rafters, then the roof area supported by a rafter is $11.18x$ sq. ft. Assuming the weight of the slate at 8 lb., the approximate weight of the rafter at 1.5 lb. per sq. ft., and the other unit weights as given in the first and second paragraphs of this article, the weights supported by a rafter are:

Slate	$11.18x \times 8$	$= 89.44x$ lb.
Sheathing	$11.18x \times 3$	$= 33.54x$ lb.
Rafter	$11.18x \times 1.5$	$= 16.77x$ lb.
Snow	$10x \times 20$	$= 200.00x$ lb.
Total vertical load		$= 339.75x$ lb.

The angle of inclination of the roof surface is 26 deg. 34 min., because its tangent is 0.5; therefore its cosine is 0.8944 and its sine 0.4472. According to the Duchemin formula the normal wind pressure is 22.3 lb. per sq. ft. and the wind load on the rafter is $11.18x \times 22.3 = 249.4x$ lb. The normal component of the dead and snow load is $339.75x \times 0.8944 = 303.9x$ lb., making the total normal load $553x$ lb. The longitudinal component of the vertical load is $339.75x \times 0.4472 = 152x$ lb.

In figuring resisting moments, dressed dimensions will be used. The resisting moment of a 2- by 4-in. rafter is $1750 \times 1\frac{5}{8} \times 3\frac{5}{8}^2/6 = 6230$ in.-lb.; of a 2- by 6-in. is 15,000; and that of a 2- by 8-in. section is 26,650 in.-lb. Computation work may be simplified by the use of a handbook on timber, such as the Southern Pine Manual, which gives the section modulus for various sizes. (See also Fig. 39*d*.) The bending moment for the normal load is $553 \times 11.18 \times 12/8 = 9280$ in.-lb. Equating the bending moment with the resisting moments in succession, the spacing of the rafters is as follows: for 2- by 4-in. section, 0.671 ft. or 8.04 in.; for 2 by 6, 19.4 in.; and for 2 by 8, 34.4 in. The spacing with 2- by 8-in. rafters would be too great for the strength of the sheathing, but 2- by 4-in. rafters would be uneconomical, so a 2- by 6-in. section will be adopted with a spacing of 18 in.

The revised total load on the rafter may next be computed: slate, 132; sheathing, 50; rafter, 26; snow, 300; or a total vertical load of 508 lb. The normal load is 454 lb. The total normal component is 828 lb. and the longitudinal component is 228 lb. The unit stress in the outer fiber due to flexure is 1622 lb. per sq. in. and that due to direct compression is 8, making the total 1630 lb. per sq. in.

Sometimes it becomes necessary to design a roof for stiffness, as well as strength, on account of the type of construction adopted. In that case the rafters are to be designed not to deflect more than, say, 1/360th of the span. As an example, the necessary computations will also be given. The formula for the deflection of a simple beam uniformly loaded is $\Delta = \frac{5Wl^3}{384EI}$; $I = \frac{bd^3}{12}$ for a beam of rectangular cross-section. The deflection allowed is $11.18 \times 12/360 = 0.373$ in.

Since experiments show that for long-continued loading the modulus of elasticity is only about one-half of its value under a short application of the same load, the dead load is doubled and added to the snow and wind loads. The total normal load is $W = [2(89.44 + 33.54 + 16.77)x + 200x] \times 0.8944 + 249.4x = 678x$ lb. Using the same section of rafter as that selected above, or 2 by 6 in., and substituting in the formula for deflection, there is obtained the equation:

$$5 \times 678x(11.18 \times 12)^3 = 0.373 \times 384 \times 1{,}600{,}000 \times 1\tfrac{5}{8} \times \frac{5\frac{5}{8}^3}{12},$$

the solution of which gives $x = 0.674$ ft. or 8.10 in. This spacing falls below the limit specified. The corresponding spacing for a 2- by 8-in. section is 19.2 in. This section would be used with a spacing of 18 in. It will be observed that the method of deflection requires 2 in. more depth than for strength.

DESIGN OF PURLINS

In Art. 60 the design of a purlin is given for the same loads and conditions as those under consideration in this article. On the basis of strength the 8- by 10-in. purlin will therefore be adopted.

Let the computations, however, be added which are required to design the purlin, so that its deflection shall not exceed 1/360th of the span, or 0.4 in. Let it be assumed that the component of the load parallel to the slope of the roof is resisted by the sheathing. It will be sufficiently accurate in getting the deflection to assume the load as uniformly distributed. The total load normal to the roof surface is 8525 lb., the dead loads having been doubled. Using the same general formula for deflection as for the rafter, the following equation is obtained:

$$5 \times 8525(12 \times 12)^3 = 0.4 \times 384 \times 1{,}600{,}000bd^3/12$$

The value of bd^3 is found to be 6220 in.^{4} Assuming $d = 9.5$ in., $b = 6.90$ in., which requires an 8- by 10-in. section, or the same section as that designed for strength.

Art. 63. Truss Loads and Stresses

For a span of 60 ft. and a spacing of 12 ft. the weight of a truss according to formula (1) in Art. 58 is $\frac{1}{2} \times 12 \times 60(1 + 0.15 \times 60) = 3600$ lb., and the weight per panel is $3600/6 = 600$ lb. Since the spacing of the rafters is 18 in., the number of rafters supported by one purlin, and hence by one panel point of the truss, is $12/1.5 = 8$. The panel loads consist of the following items:

Slate	8 × 132 =	1056 lb.
Sheathing	8 × 50 =	400 lb.
Rafters	8 × 26 =	208 lb.
Purlin	1 × 214 =	214 lb.
Truss per panel	=	600 lb.
Dead panel load		2478 lb.
Snow panel load	8 × 300 =	2400 lb.
Total		4878 lb.
Ceiling panel load	10 × 12 × 20 =	2400 lb.
Wind panel load	8 × 374 =	2992 lb.

Including the half upper panel load at the end due to dead and snow load, but excluding the half panel load due to the ceiling, the reaction at the support of the truss is $\frac{1}{2}(6 \times 4878 + 5 \times 2400) = 20{,}630$ lb. The stresses in the members of the truss are determined by the graphic method as given in Roofs and Bridges, Part II, Chapter II, and the values are marked on the diagram in Fig. 63*a*. The total length of the load line for the upper panel dead and snow loads is $6 \times 4878 = 29{,}270$ lb.; and that for the ceiling load is 5 × 2400 = 12,000 lb. These two load lines are laid off with their centers coinciding. The dead and snow load

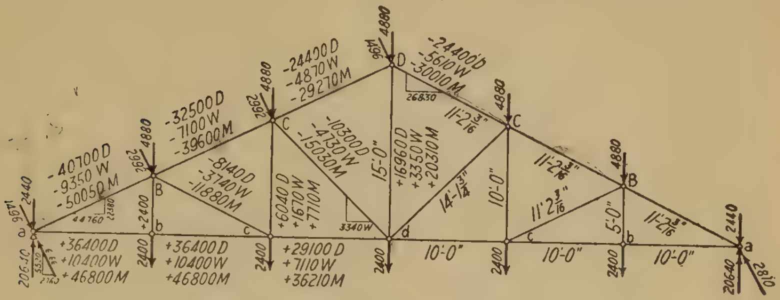

a
B
b
C
c
D
d
C
c
B
b
a
2440
967
4880
2662
4880
2662
4880
967
4880
4880
2440
-40700 D
-9350 W
-50050 M
44760
22380
-32500 D
-7100 W
-39600 M
-24400 D
-4870 W
-29270 M
-24400 D
-5610 W
-30010 M
26830
-8140 D
-3740 W
-11880 M
-10300 D
-4730 W
-15030 M
+2400
+6040 D
+1670 W
+7710 M
3340 W
15'-0"
+16960 D
+3350 W
+20310 M
+36400 D
+10400 W
+46800 M
+36400 D
+10400 W
+46800 M
+29100 D
+7110 W
+36210 M
20610
3330
2760
2400
2400
2400
2400
2400
11'-2 3/16"
11'-2 3/16"
11'-2 3/16"
11'-2 3/16"
14'-1 3/4"
10'-0"
5'-0"
10'-0"
10'-0"
10'-0"
20640
2810

Fig. 63*a*.—Panel Loads, Stresses, and Lengths of Members.

stresses may be found by a single diagram, since the minimum stresses are not required.

Art. 64. Sections of Truss Members

In the accompanying table the designation of the members refers to the notation shown in Fig. 63*a*. The lengths, center to center of panel points, are computed with the aid of a table of squares. The maximum stresses are taken from Fig. 63*a*. The section dimensions of the compression members are determined in the manner explained in Art. 52.

The values of $\frac{l}{d}$ are the ratios of length to least diameter, both dimensions being expressed in inches. The values of f are the allowable unit stresses. The section areas for the rods, both required and furnished, are those at the root of the thread, since only plain rods are used. Since two of the rods are too short to make upset ends economical, it is considered best to use rods without upset ends throughout. The section area furnished for the lower chord cannot be determined until after the end joint is designed.

Except in very large trusses, it is customary to make the upper chord of uniform section throughout. In large spans the width is kept the same, but the depth is reduced in some of the upper panels. Or, in case the chord is built up of three pieces bolted together, the middle piece may be omitted in the upper panels, and packing blocks inserted between the outer pieces at the panel points, cast-iron separators being used at intermediate points where they are bolted together. Planks are less expensive than large dimension timbers, and have fewer hidden defects in the wood; but as they must be bolted together at close intervals, they require extra cost in bolts and labor which may counterbalance or exceed the initial saving in lumber. Furthermore, since the strength of a composite column is materially less than that of a solid timber of the same gross section, it is not economical generally to use a built-up chord member to resist compression.

STRESSES AND SECTIONS

Member	Length,		Maximum Stress, Lb.	Size, In.	$\frac{l}{d}$	f Lb. per Sq. In.	Section Required, Sq. In.	Area Furnished, Sq. In.
	Ft.	In.						
aB	11	$2\frac{3}{16}$	−50,050	8 × 8	17.9	1,110	45.1	56.3
BC	11	$2\frac{3}{16}$	−39,600	8 × 8	17.9	1,110	35.7	56.3
CD	11	$2\frac{3}{16}$	−29,270	8 × 8	17.9	1,110	26.4	56.3
Bc	11	$2\frac{3}{16}$	−11,880	6 × 6	23.85	770	15.45	31.6
Cd	14	$1\frac{3}{4}$	−15,030	6 × 6	30.2	480	31.5	31.6
Bb	5	0	+ 2,400	$\frac{5}{8}\phi$	....	18,000	0.133	0.202
Cc	10	0	+ 7,710	1ϕ	....	18,000	0.428	0.551
Dd	15	0	+20,310	$1\frac{1}{2}\phi$	....	18,000	1.128	1.294
ac	20	0	+46,800	....	....	1,750	26.7	
cd	10	0	+36,210	....	....	1,750	20.7	

The upper chord should be as nearly square in section as possible, as this is the best section for a column. When the dimensions of the cross-section are unequal, it depends upon the quality of the wood whether the long or the short side is to be placed horizontally. Since the entire horizontal component of its stress has to be transmitted into the lower chord, the two chords should have the same width. If the chords are too narrow, the lugs of the end shoes must be deeper, and therefore require thicker material. When steel forged shoes are employed, the cost increases rapidly with any decrease in width.

For trusses of ordinary dimensions, it improves the appearance to make all the struts the same width as the chord, giving flush surfaces at the joints. In large trusses it is more economical to reduce the width from the middle toward the ends, but it is not desirable to have too many different sections. Occasionally, the struts are built of two sticks, united by bolts and filler pieces; but they must be designed as composite columns in that case (Art. 50). Sometimes the sections of one or more struts have to be increased on account of the bearing against the side of the upper chord.

When a single tie-rod exceeds $1\frac{1}{2}$ or $1\frac{5}{8}$ in. in diameter, it is usually preferable to substitute two rods of smaller diameter, on account of the distribution of the pressure under the washers.

PROBLEM 64. In case a steel rod with upset ends were substituted for the plain rod *Dd*, find the diameter of the rod and of its upset ends.

ART. 65. DESIGN OF JOINT DETAILS

The details at the joints to be designed include the washers, and the depths of notches or indents to resist the longitudinal components of the stresses in the struts, with respect to the chords. It is also necessary to investigate the bearing surfaces of the struts on the sides of the chords, to see whether the section of the strut has to be increased or a bearing block provided. To determine the allowable unit stresses on surfaces inclined to the fibers the Hankinson formula as given in Art. 91 will be used.

DESIGN OF WASHERS

JOINT *B*. It will be most convenient to cut a triangular notch in the top of the chord, to receive the bearing of a horizontal washer. In order to cut away as little as possible of the bearing area for the purlin at this panel point, a square steel-plate washer is selected. The angle between the bearing surface and the fibers is $26\frac{1}{2}$ deg. Using Fig. 91*a*, the value p/q being $1713/380 = 4.50$. it is found that $K = 0.265$; hence $s = 1713 \times 0.265 = 450$ lb. per sq. in. The net bearing area required is $2400/450 = 5.33$ sq. in. The diameter of the rod is $\frac{5}{8}$ in., and of the hole in the washer $\frac{11}{16}$ in.; hence the gross area is $5.33 + 0.37 = 5.70$ sq. in., which requires a 2- by 3-in. washer. The thickness required may be found by treating the washer as a beam, the upward force coming from the timber and the downward force coming from the nut, which has a short diameter of $1\frac{1}{16}$ in. The equation then is

$\frac{1}{6} \times 18{,}000 \times (2 - \frac{11}{16})t^2 = \frac{2400}{2}\left(\frac{3}{4} - \frac{1.07}{4}\right)$, or $t = 0.384$ in., say, $\frac{7}{16}$ in.

JOINT *C*. Net bearing area = 7710/450 = 17.13 sq. in.; diameter of rod, 1 in.; diameter of hole, $1\frac{1}{16}$ in.; gross area = 17.13 + 0.89 = 18.02 sq. in.; size of washer required 4 by $4\frac{1}{2}$ in.; thickness required $\frac{9}{16}$ in.

JOINT *D*. Net bearing area = 20,310/450 = 45.1 sq. in.; diameter of rod, $1\frac{1}{2}$ in.; diameter of hole, $1\frac{5}{8}$ in.; gross area, 45.1 + 2.07 = 47.17 sq. in., size of washer required, $6\frac{1}{2}$ by $7\frac{1}{2}$ in.; thickness is given by the equation

$$\frac{1}{6} \times 18{,}000(6.5 - 1.62)t^2 = \frac{20{,}310}{2}\left(\frac{7.5}{4} - \frac{2.37}{4}\right), \text{ or } t = 0.94 \text{ in.}$$

A thickness of 1 in. will be used.

JOINT *b*. The bearing of this washer is perpendicular to the fibers, hence the allowable unit stress is 380 lb. Net bearing area = 2400/380 = 6.32 sq. in.; using a cast-iron ogee washer, a $3\frac{1}{8}$-in. washer (Table 2*c*) furnishes a bearing area of 7.23 sq. in.

JOINT *c*. Net bearing area = 7710/380 = 20.3 sq. in.; a $5\frac{1}{4}$-in. diameter ogee washer of the type shown in Table 2*c* furnishes an area of 20.66 sq. in.

JOINT *d*. Net bearing area = 20,310/380 = 53.4 sq. in.; diameter of bolt, $1\frac{1}{2}$ in.; diameter of hole $1\frac{5}{8}$ in.; gross area 53.4 + 2.07 = 55.47 sq. in.; size of washer required $7\frac{1}{2}$ by $7\frac{1}{2}$ in.; thickness, $\frac{7}{8}$ in.

DESIGN OF OTHER JOINT DETAILS

JOINT *B*. The load from the strut *Bc* will be carried into the upper chord by notching the strut into the chord, the two bearing surfaces being at right angles to each other. Each surface will be designed to carry the component of the stress normal to it. Assuming the depth of indent to be $2\frac{1}{2}$ in., the angle between the indent and the strut is $57\frac{1}{2}$ deg., and the allowable unit compression (Art. 91) is 855 lb. per sq. in. The depth of indent needed depends on the allowable bearing for the strut. The component of the stress perpendicular to the indent is 10,000 lb., and the net width of indent, taking out a $\frac{11}{16}$-in. hole for the $\frac{5}{8}$-in. rod, is 5.62 − 0.69 = 4.93 in. Hence the depth of indent required is 10,000/(4.93 ×

855) = 2.37 in., say $2\frac{1}{2}$ in. The angle which the other bearing makes with the upper chord, the chord being weaker on this plane than the strut, is $20\frac{1}{2}$ deg. and the allowable unit stress is 425 lb. per sq. in. The length of bearing required is 6300/(5.62 × 425) = 2.64 in. As it measures $6\frac{1}{2}$ in., no revision is necessary.

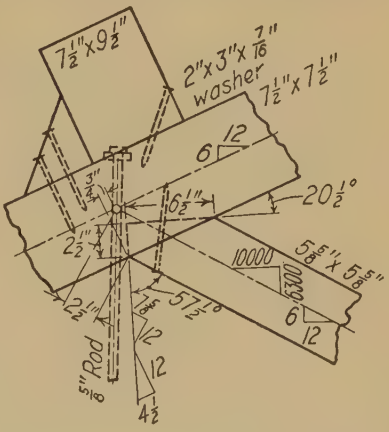

$7\frac{1}{2}$"x$9\frac{1}{2}$"
2"x3"x$\frac{7}{16}$"
washer
$7\frac{1}{2}$"x$7\frac{1}{2}$"
12
6
$\frac{3}{4}$"
$6\frac{1}{2}$"
$20\frac{1}{2}$°
$2\frac{1}{2}$"
10000
6300
$5\frac{5}{8}$"x$5\frac{5}{8}$"
6
12
$2\frac{1}{2}$"
$57\frac{1}{2}$°
$1\frac{1}{8}$
12
12
$4\frac{1}{2}$
$\frac{5}{8}$" Rod

FIG. 65*a*.—Joint *B*.

JOINT *C*. A preliminary design of the joint shows that it will be necessary to increase the size of the strut *Cd* from $5\frac{5}{8}$ by $5\frac{5}{8}$ in. to $5\frac{5}{8}$ by $7\frac{1}{2}$ in. before enough bearing area can be furnished. Assuming $2\frac{3}{4}$ in. for the shorter side of the indent, the angle of the cut with the axis of the strut is 46 deg. and the allowable bearing is 635 lb. per sq. in. The stress component is 10,750 lb. and the net width of bearing is (7.50 − 1.06) = 6.44 in., the diameter of the hole being $1\frac{1}{16}$ in. The required depth of indent is 10,750/(6.44 × 635) = 2.63, say $2\frac{3}{4}$ inches. For the longer side of the indent the angle with the upper chord is $27\frac{1}{2}$ deg., the unit allowable bearing, 460 lb., and the required length of side, 10,500/(7.5 × 460) = 3.05 in., $5\frac{1}{2}$ in. is furnished.

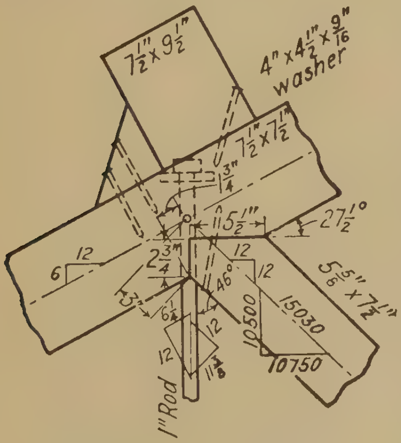

$7\frac{1}{2}$"x$9\frac{1}{2}$"
4"x$4\frac{1}{2}$"x$\frac{9}{16}$"
washer
$7\frac{1}{2}$"x$7\frac{1}{2}$"
$\frac{3}{4}$"
$5\frac{1}{2}$"
$27\frac{1}{2}$°
12
6
$2\frac{3}{4}$"
12
46°
12
15030
10500
10750
$5\frac{5}{8}$"x$7\frac{1}{2}$"
3"
$6\frac{1}{4}$
12
12
$1\frac{1}{8}$
1" Rod

FIG. 65*b*.—Joint *C*.

JOINT *D*. The bearing surface between the upper chord members is inclined at an angle of $63\frac{1}{2}$ deg. with the fiber, making the allowable unit compression 1010 lb. per sq. in. Since the horizontal component of the maximum chord stress is 26,830, the vertical bearing area needed is 26,830/1010 = 26.6 sq. in. Since the bolt hole reduces the horizontal width of the bearing by $1\frac{5}{8}$ in., the height required is $26.6/(7.5 - 1.62) = 4.53$ in. As detailed, $6\frac{3}{4}$ in. is provided.

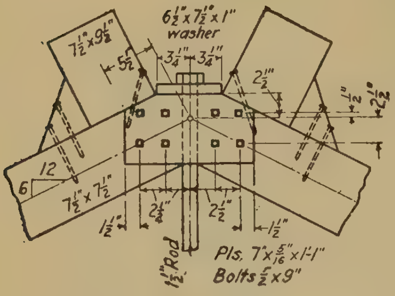

6½"x7½"x1"
washer
7½"x9½"
5½
3¼"
3¼"
2½"
½"
2½"
12
6
7½"x7½"
1½"
2¼"
2½"
1½"
1½" Rod
Pls. 7"x5/16"x1'-1"
Bolts ⅝x9"

FIG. 65*c*.—Joint *D*.

JOINT *d*. An angle block will be used for the bearing of the lower end of *Cd*, the grain of the angle block being horizontal. The bearing of the strut on the block will be at an angle of 45 deg. to the fibers, hence the allowable unit bearing as found from Fig. 91*a* is $s = 1713 \times .365 = 625$ lb. per sq. in. The area required is $15{,}030/625 = 24.0$ sq. in.; that furnished is $5\frac{5}{8} \times 7\frac{1}{2} = 42.2$ sq. in. The indent must provide sufficient bearing against the angle block to resist the horizontal component of the greatest difference between the stresses in the diagonal struts that bear against the block. This difference is 3340 lb., or the horizontal component of the greatest wind stress in one strut. The depth of the indent must then be at least $3340/(1713 \times 6.62) = 0.35$ in.; but it is better to make it $\frac{1}{2}$ in. The bearing on the base of the block must resist the full stress in the vertical rod or 20,310 lb. The required bearing area is therefore $20{,}310/380 = 53.4$ sq. in. As the block is 18 in. long and $7\frac{1}{2}$ in. wide, there is a large excess area furnished. Dowels are inserted to hold the struts in position, as shown in Fig. 65*d*.

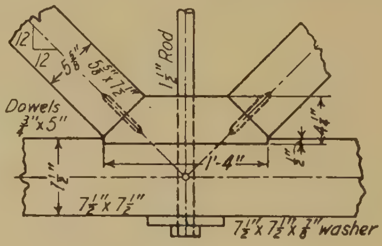

12
12
5⅝"
5⅝"x7½"
1½" Rod
Dowels
¾"x5"
4¼"
½"
1'-4"
7½"
7½"x7½"
7½"x7½"x⅞"washer

FIG. 65*d*.—Joint *d*.

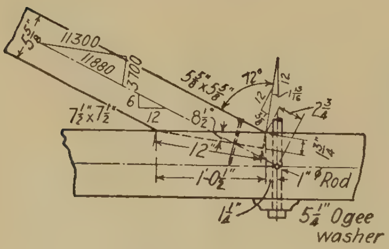

5⅝"
11300
11880
3700
5⅝"x5⅝"
72°
12
12
1 13/16
2¾
6
12
7½"x7½"
8½"
12"
¾
1'-0½"
1" Rod
1¼"
5¼" Ogee
washer

FIG. 65*e*.—Joint *e*.

JOINT *c*. In a manner similar to that explained for joint *B* the depth of the indent is found to be 1.57 in., when $1\frac{3}{4}$ in. is used, the angle of the indent with the fibers of the strut being 72 deg. The angle which the longer side makes with the fibers of the lower chord is $8\frac{1}{2}$ deg., making the required length of bearing 1.74 in.

ART. 66. DESIGN OF END JOINT

The order of design given in Art. 61 applies more particularly to the types shown on Plates I and II. An alternative design of that type for the truss under consideration in this chapter is presented later in this article. Since steel-plate connections are efficient and make simple details, the first design is to be one containing steel side plates with flats riveted to them, which act in exactly the same manner as in the tabled fish-plate joint. On each chord the tables which receive the compression are placed perpendicular to the direction of the stress, so that the bearing is squarely on the ends of the fibers. This design avoids all eccentricity in the joint or secondary flexure in either chord member.

The maximum stress in the upper chord is 50,050 lb. The bearing area required is $50{,}050/1713 = 29.20$ sq. in., and the required shearing area is $50{,}050/210 = 238$ sq. in. Assuming two tables on each side of the chord, the thickness of the flats composing the tables is $29.20/(4 \times 7.5) = 0.967$ in., say 1 in. The clear distance between the tables equals the shearing length for one table, making it $238/(4 \times 7.5) = 7.93$ or $8\frac{1}{4}$ in., which allows for the bolt holes to be deducted from the shearing surface.

The pressure against one table is 12,510 lb., which, according to a handbook, requires three $\frac{3}{4}$-in. rivets in single shear to resist it, the unit allowable shear being 12,000 lb. per sq. in. In order to provide sufficient bearing area against the rivets, the large connecting plates must be $\frac{5}{16}$ in. thick. The width of the flats should be at least $2\frac{1}{2}$ in. (Art. 22), but to reduce the lateral pressure due to the moment of rotation on the tables let the width be increased to 3 in. The outer rivets will be placed $1\frac{1}{4}$ in. from the end of the flat, making the spacing between the rivets $2\frac{1}{2}$ in.

Each plate has to be investigated as a column between the sections where the bolts hold it in position. The radius of gyration of the plate is $r = 0.289 \times \frac{5}{16} = 0.0903$ in.; and since the length between centers of tables is 11.25 in.,

$$\frac{l}{r} = 11.25/0.0903 = 124.6,$$

which exceeds the allowable limit of 120. The thickness of the plate, therefore, will be increased to ⅜ in.

The moment of rotation on each table is 12,510(0.188 + 0.50) = 8600 in.-lb. The stress in each of the two bolts next to the table which are 3½ in. from the bearing surface is 8600/(2 × 3.5) = 1230 lb. This requires a bolt only $\frac{7}{16}$ in. in diameter, but since the bolts have additional duty in holding the plates in position, the diameter is increased to ½ in. To hold the upper edges of the plate in contact with the timber two track spikes $\frac{5}{16}$ by 2½ in. may be employed.

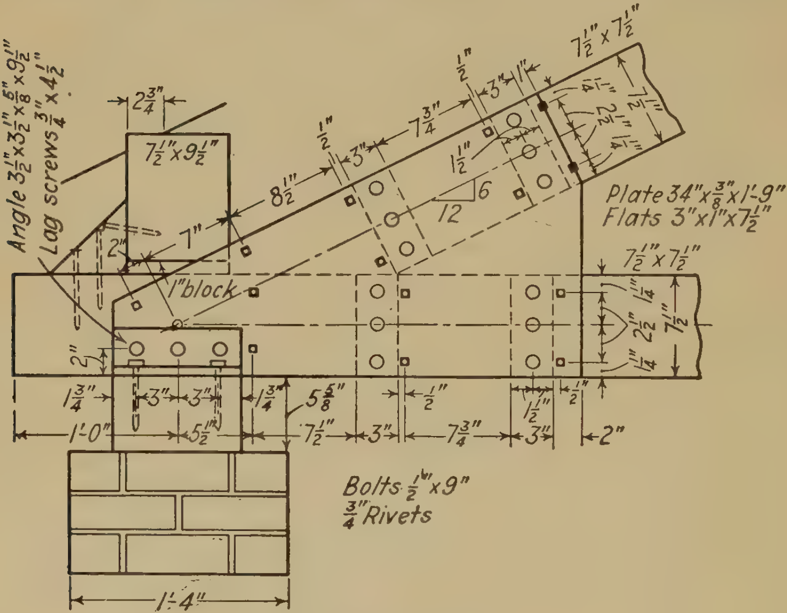

Angle 3½"×3½"×⅜"×9½"
Lag screws ¾"×4½"
2¾"
7½"×9½"
2"
1"
1"block
8½"
½"
3"
7¾"
½"
3"
1"
1½"
6
12
7½"×7½"
2½"
1½"
Plate 34"×⅜"×1'-9"
Flats 3"×1"×7½"
7½"×7½"
1¾"
2½"
7½"
1¾"
2"
1¾"
3"
3"
5½"
1¾"
5⅝"
½"
½"
½"
1'-0"
7½"
3"
7¾"
3"
2"
Bolts ½"×9"
¾"Rivets
1'-4"

FIG. 66*a*.—End Joint *a*.

In a similar manner the connections with the lower chord timber are designed. Assuming a depth of 7½ in. for the timber, the required thickness of flats is 0.91 in., but they will be made 1 in. thick, the same as for the upper chords. The length of the shearing surface required is 7.43 or 7½ in. Let this distance also be made the same as for the upper chords, or 8¼ in. In this case the bolts must be placed on the opposite side of the tables, and hence the end table is placed farther from the edge of the plate than at the upper chord timber. For bolt holes $\frac{9}{16}$ in. in diameter, the net section of the 8- by 8-in. timber is $(7.5 - 2 \times 1)\ (7.5 - 2 \times 0.562) = 29.60$ sq. in., which is in excess of the 26.7 sq. in. required. Other dimensions and details are given in Fig. 66*a*.

It is also found by computation that the lower chord has sufficient excess of strength at the middle of the end panel to carry the future ceiling load as a uniform load between panel points. As the same section is continued throughout and the stresses are less in the other panels, the entire ceiling load may be supported in this manner, if desired.

For the type of joint shown in Fig. 66*b* the entire horizontal component of the stress in the upper chord is assumed to be taken by the lugs which engage notches in the lower chord, and also by the shoe plate at the toe. Since the pressure of the plate on the sides of the fibers increases their resistance, the unit compressive stress at the toe may be taken the same as on the ends of the fibers. The depth of toe needed is therefore $44{,}760/(1713 \times 7.5) = 3.49$ in., say 4 in. The horizontal bearing required is $22{,}380/380 = 58.9$ sq. in., and its length is $58.9/7.5 = 7.86$ in., or 8 in. to provide for the hole.

If two lugs are employed, their depth of bearing must be $44{,}760/(2 \times 1713 \times 7.5) = 1.74$ in., or $1\frac{3}{4}$ in. The bending moment for the lug, treated as a cantilever under uniform load, is $22{,}380 \times 0.875 = 19{,}580$ in.-lb. The resisting moment of the lug is $18{,}000 \times 7.5 \times \frac{d^2}{6}$, and when equated to the bending moment gives $d = 0.97$ or 1 in.

The net shearing surface required between the lugs is $22{,}380/210 = 106.6$ sq. in. Allowing 1 sq. in. for the bolt hole, the required distance between lugs is $(106.6 + 1)/7.5 = 14.35$ in. This same distance is needed from the outer lug to the end of the chord. Since the horizontal component of the wind reaction can be transmitted to the chord through the bolster and its key, the net tensile area at the inner lug needs only to be $44{,}760/1750 = 25.6$ sq. in. The net depth of the chord is then $25.6/7.5 = 3.42$ in., making the gross depth $3.42 + 1.75 = 5.12$, or $5\frac{5}{8}$ in. dressed thickness.

The inclined bolts shown in Fig. 66*b* are not designed to take any of the horizontal component of the stress of the upper chord. Their function is to hold the parts firmly together and to offset the moment in the upper chord caused by the eccentric framing into the lower chord. The moment of the forces on the lugs about the intersection of the center line of the upper chord with the resultant of the horizontal bearing is 44,760 × 3.12 = 139,500 in.-lb. Using $\frac{7}{8}$-in. bolts, the strength of each bolt is 0.419 × 18,000 = 7540 lb.; and the moment of resistance is 7540(15 + 3) = 135,700 in.-lb. The net bearing area of the washer on the bolster is 7540/450 = 16.75 sq. in., since the allowable stress on the section making an angle of 26½ deg. with the fibers is 450 lb. per sq. in.

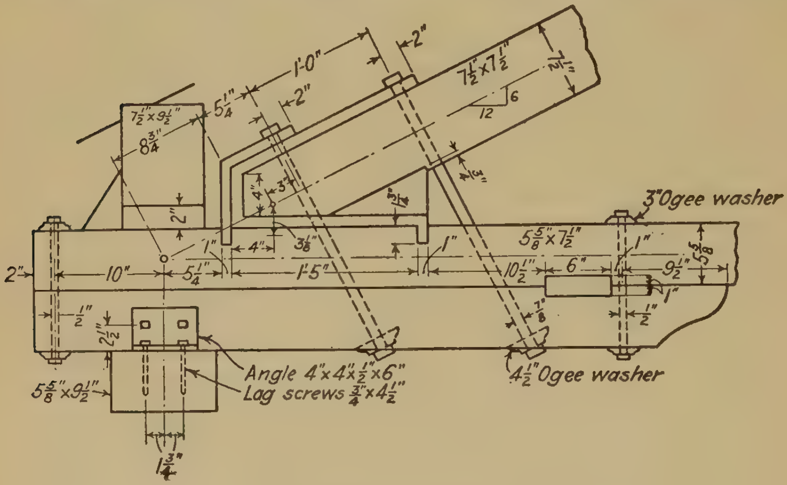


FIG. 66b.—End Joint a.

A 4½-in. ogee washer will be used, which furnishes a bearing of 15 sq. in. Any larger washer cuts too far into the bolster.

A 5⅝- by 7½-in. bolster is added to the end joint, both to avoid cutting the lower chord on account of the washers for the inclined bolts and to stiffen the chord timber, which is subject to bending on account of eccentricity in the applied forces. The key connecting the bolster should be designed to transmit the horizontal component of the full safe tensile stress in the inclined bolts as well as the horizontal component of the reaction at the support due to wind pressure on the truss. This gives a force equal to (2 × 7540 + 6170) × 0.4472 = 9500 lb., hence the depth of the key is 2 × 9500/(1713 × 7.5) = 1.478 in., say 2 in. The length needed to resist the shear parallel to the fibers and that to resist the moment of rotation, are found by the method explained in Art. 17 to be 6.03 and 4.47 in. Let the length be made 6 inches. The size of bolt required to resist the vertical reaction of the key is only slightly over $\frac{1}{2}$ in., so this will be used. A 3-in. ogee washer is required.

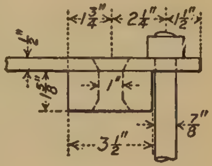


Fig. 66*c*.

The preceding design may be modified so as to reduce the cost materially, by substituting for the inner lug a flat riveted to the plate as illustrated in Fig. 66*c*. In this the thickness of the plate is not determined by flexure and may therefore be greatly reduced in thickness. Since only three rivets can be inserted in a single row across the chord, each rivet must resist a stress in single shear of 22,380/3 = 7460 lb. This requires a 1-in. rivet. The plate must have sufficient thickness for the same stress in bearing on the rivet, which must therefore be $\frac{5}{16}$ in., but this is increased to $\frac{1}{2}$ in. because the rivet heads have to be countersunk. The width of the flat should be made $3\frac{1}{2}$ in. (Art. 22). The rivets will be spaced 3 in., the edge distance being $1\frac{1}{2}$ in.

Problem 66. Design a forged steel shoe and its connecting bolts which shall provide three bearing surfaces for the timber instead of two.

Art. 67. Supports and Splices

The purlins may be held in position by various means. For the intermediate purlins the simplest connection consists of a wooden triangular block and spikes. The component along the chord of the maximum pressure from the purlin is 1730 lb. This requires six 60d wire nails or spikes, since Table 6*a* gives the safe working strength for one nail as 323 lb. Two of these may be put through the upper side of the purlin into the chord timber. Two or three nails will be driven through the block into the purlin. The purlin need not then be notched over the chord. (The weakening effect of notching is discussed in Art. 40.) The intermediate purlins require a bearing area of 6815/380 = 17.9 sq. in. The area furnished is 7.5 × 7.5 = 51.2 sq. in.

The vertical component of the maximum reaction is 26,160 lb. which requires a bearing area of 26,160/380 = 68.7 sq. in. For a lower chord width of $7\frac{1}{2}$ in., the width of the wall plate must be $68.7/7.5 = 9.16$ in., or $9\frac{1}{2}$ in. dressed thickness. Where the width of the wall plate is unduly high it may be reduced by inserting a block of stronger timber and using a wall plate of stronger timber.

If the truss is supported by a wall of common brickwork, the allowable pressure per square inch is about 100 lb. A bearing surface is required of $26,160/100 = 261.6$ sq. in., and for a width of $9\frac{1}{2}$ in. the length is 27.5 or 28 in. The thickness for the second type of joint may be approximately computed by treating each end of the 28-in. wall plate as a cantilever under uniform load. The result is 4.93 in., say, $5\frac{5}{8}$ in. dressed thickness.

No definite computation can be made regarding the uplifting effect of wind. Two anchor bolts $\frac{3}{4}$ in. in diameter and 24 in. long will give ample strength for the purpose.

In the first design the upper chord load is carried to the steel plates, the horizontal component then going from the plates into the lower chord. The vertical component will be carried directly into the wall plate, through angles. The load to be carried is 22,380 lb., hence the bearing area required is $22,380/380 = 58.8$ sq. in. The area furnished by two $3\frac{1}{2}$- by $3\frac{1}{2}$- by $\frac{5}{8}$-in. angles is $2 \times 3.5 \times 9.5 = 66.5$ sq. in. The bending moment per inch of angle due to the upward forces and about the beginning of the fillet is $\frac{380 \times 2.5^2}{2} = 1187$ in.-lb.; and the strength of the section is $\frac{1}{6} \times 18,000 \times \frac{5}{8} \times \frac{5}{8} = 1172$ in.-lb. Six $\frac{3}{4}$-in. rivets will carry a load of 31,800 lb.

The truss is held in position on the wall plate by lag screws, as shown in Fig. 66*a*. The horizontal force to be resisted is the horizontal component of the reaction due to wind pressure which equals 2760 lb. Four $\frac{3}{4}$-in. lag screws will take 4120 lb. (Art. 14.) In the alternative design the bolster may be notched down over the wall plate to resist this pressure.

The splice in the upper chord should be made near the quarter point of the panel length, which is assumed to be the location of the point of inflection in the chord timber acting as a column. The half lap joint is an effective one for the purpose. and is given the proportions indicated in Art. 31.

If the truss is to be hoisted into position by a derrick, as is sometimes the case, the splice at the peak has to resist considerable tension, and should be designed in all its details to resist the computed erection stresses. An effective joint may be secured by metal plates and bolts. The metal plates may be replaced by wooden planks if desired.

The splice in the lower chord is preferably placed at the center of the span, since the length of each stick will not exceed a car length. If, however, the available timber is not so long, the joint should be placed in the panel *cd* (Fig. 63*a*) as close to the panel point *c* as convenient, due regard being paid to commercial lengths of lumber. As steel connecting plates were adopted for the end joint in the first design, a tabled fish-plate joint with steel plates may be appropriately selected. For the alternative design, wooden fish plates may be adopted. Since complete designs for fish-plate joints are given in Chapter II, no details regarding the lower chord splice will be given in this article.

ART. 68. ANALYSIS OF WEIGHT

The following bills of material include one roof truss and a single bay of the roof, or that portion which is supported by one intermediate truss. All the wood is Douglas fir, the weight being taken as 3 pounds per foot board measure.

The weight of the overhanging portion of the roof including the cornice is 1450 lb.; hence the net weight for six panels of the roof truss is 10,984 lb. On adding the weight of the truss the average dead panel load is $(10{,}984 + 3157)/6 = 2356$ lb. The panel dead load assumed in finding the stresses in the roof truss exclusive of the ceiling is 2478 (Art. 63), which is on the safe side.

The weight of the roof truss according to the formula used is 3600 lb., whereas the actual weight is only about 3150 lb. For the material, loading and unit stresses adopted for this truss, the second constant in the parenthesis of the formula should be 0.13 instead of 0.15.

The theoretic weight (Art. 58) of this truss is 2317 lb., consisting of 2163 lb. of lumber and 154 lb. of steel rods. To obtain the actual weight of the truss members and of all details it is therefore necessary to add 36.1 percent to the theoretic weight. The weight of metal, including rods, plates, washers, angles, bolts and screws is 806 lb., which is 25.6 percent of the actual total weight.

MATERIAL FOR ONE TRUSS

Designation	Number of Pieces	Section, Dimension in Inches	Length, Feet and Inches	Feet, Board Measure, Nominal	Weight, Pounds, Dressed
Timber:					
Upper chord	2	8 × 8	22– 0	235	618
Upper chord	2	8 × 8	12– 3 [14]	149	344
Lower chord	2	8 × 8	30– 0	320	842
Strut *Bc*	2	6 × 6	10–10¾ [12]	72	172
Strut *Cd*	2	6 × 8	13– 1 [14]	112	276
Block at *a*	2	2 × 8	7½	2	4
Block at *a*	2	6 × 6	7½	4	9
Blocks at *B* and *C*	2	6 × 6	6	3	8
Block at *d*	1	4¾ × 7½	1– 4	4	12
Wall plate	2	6 × 10	2– 4	23	62
Total				924	2347
				Weight Per Foot	Weight Pounds
Steel Rods:					
Rod *Bb*	2	⅝ϕ	5– 9	1.043	12
Rod *Cc*	2	1 ϕ	10–10	2.67	58
Rod *Dd*	1	1½ϕ	15–11½	6.008	96

		Weight, Pounds	
Nuts:			
4 for ⅝-in. rod; 4 for 1-in. rod; 2 for 1½-in. rod		6	
Plate Washers:			
Two at *B*, 2 × 3 × $\frac{7}{16}$ in.; two at *C*, 4 × 4½ × $\frac{9}{16}$ in.; one at *D*, 6½ × 7½ × 1 in.; one at *d*, 7½ × 7½ × ⅞ in.; total weight		40	
Cast-iron Ogee Washers:			
Two at *c*, 5¼ in. in diameter; two at *b*, 3 in. in diameter; eight for half-lap joint, 4 in. in diameter		16	
Steel Plates:			
At *a*, 4 plates, 34 × ⅜ in. × 1 ft. 9 in	@ 75.8	303.2	
16 flats, 3 × 1 × 7½ in.	@ 6.4	102.3	
4 angles, 3½ × 3½ × ⅝ in. × 9½ in.	@ 10.8	43.2	
At *D* 2 plates 7 × $\frac{5}{16}$in. × 1 ft. 1 in.	@ 8.04	16.1	
For splice in lower chord:			
2 plates 7½ × $\frac{5}{16}$ in. × 3 ft. 2 in	@ 25.3	50.6	
8 flats 2½ × ¾ × 7½ in.	@ 3.98	31.8	
Total			609.2
Steel Bolts including Nuts:			
At *a*, 24 bolts, ½ × 9 in.; at *D*, 8 bolts, ½ × 9 in.; for half-lap joint, 4 bolts, ¾ × 10½ in.; for splice in lower chord, 10 bolts, ⅜ × 8½ in.; total weight		28	
Lag Screws:			
At *a*, 8¾ × 4½ in.; at *B*, 2½ × 6 in.; at *C*, 2½ × 6 in.; at *c*, 2¼ × 5 in.; total weight		7	
Total weight of one truss			3157

Weight of One Bay of Roof, 12 Ft. Long

	Pounds
8 Purlins, 7½ × 9½ in. × 12 ft., 570 ft. b.m.	1710
12 Rafters, 1⅝ × 5⅝ in. × 23 ft. 10¾ in. [24 ft.], 286 ft. b.m.	858
12 Rafters, 1⅝ × 5⅝ in. × 14 ft. 6¾ in. [16 ft.], 174 ft. b.m.	522
Sheathing, 1 in. thick, 12 ft. × 76 ft., 912 ft. b.m.	2,736
Sheathing, etc., for cornice, 210 ft. b.m.	630
Slate, 12 × 20 × $\frac{3}{16}$ in., 8½ in. exposed when laid, 912 sq. ft., @ 6½ lb.	5,928
Roofing nails, 4d galvanized	19
Nails for rafters, 10d, 3 lb.; for purlins, 60d, 5 lb.	8
Nails for sheathing and cornice, 8d, 21 lb.; 20d, 2 lb.	23
Total weight	12,434

ART. 69. ESTIMATE OF COST

The approximate cost of the roof truss according to the first design, in which vertical steel plates with flats riveted to them are employed at the end joints, and similar fish plates are used for the splice in the lower chord, is estimated as follows:

Lumber, 921 ft. b.m. @ 5 cents	$46.05
Steel rods and nuts, 172 lb. @ 6 cents	10.32
Steel plates, angles and washers, 587 lb. @ 6 cents	35.22
Steel bolts and nuts, 28 lb. @ 6 cents	1.68
Lag screws, 7 lb. @ 10 cents	.70
Cast-iron ogee washers, 16 lb @ 5 cents	.80
Framing and erection, 921 ft. b.m. @ 4½ cents	41.44
Total	$136.21

In the following estimate of cost for material in one bay of the roof, one-tenth is added to the superficial area to allow for waste in the sheathing, and one-fifth is added to the lumber in the cornice. The cost of carpenter work includes that of nails.

Lumber, 2375 ft. b.m. @ 5 cents	$118.75
Framing and placing lumber, 2375 ft. b.m. @ 3 cents	71.25
Slate, 9.5 square, @ $12.00	114.00
Labor for slating, 48 hr. @ 80 cents	38.40
Roofing nails, 19 lb @ 6 cents	1.14
Total	$343.54

The total cost, therefore, of one truss and the roof which it supports is $479.75.

The preparation of bills of material in a proper form, and of detailed estimates of cost, is a subject to which the young designer should give especial study. There is comparatively little material of this kind published in the form in which it is used in practice. As an interesting example of a comparative estimate for three types of construction, refer to an article on Round-house Framing by R. D. Coombs, in Transactions of American Society of Civil Engineers, vol. 55, page 157, December, 1905; or Railroad Gazette, vol. 38, page 624, June 9, 1905. For an example of the analysis of cost for a structure, but one in which lumber was used only for falsework and forms, see an article on Arch Rib Bridge of Reinforced Concrete at Grand Rapids, by George Jacob Davis, Eng. News, vol. 55, page 321, March 22, 1906.

ART. 70. DETAIL DRAWINGS

Because many of the details for the design of the roof truss made in this chapter have been illustrated in preceding articles, the working drawing will be omitted. In its stead are given Plates I and II, which are reproduced by lithography from the original detail drawings prepared by two students as a part of their regular class work.

The design of C. L. Todd is for a span of 82 ft., with a rise equal to one-fourth of the span, the trusses being spaced 14 ft. It will be observed that the steel shoe plate has three bearing surfaces in the lower chord, the middle one being provided by an angle riveted to the plate. The general arrangement of details, the dimensions required for construction, etc., are all clearly shown and require no additional description.

The design of R. M. Bowman is for a span of 80 ft., with a rise of one-fourth of the span, the trusses being spaced 12 ft. apart. The truss is of the Belgian type, like Fig. 57*b*, in which the struts are perpendicular to the upper chord. The beveled washers shown are all of the same type. The flat riveted to the steel shoe plate is connected by two rows of staggered rivets. In both cases the designs were made for longleaf yellow pine, the unit stresses adopted differing somewhat from those for the same wood used in this textbook.

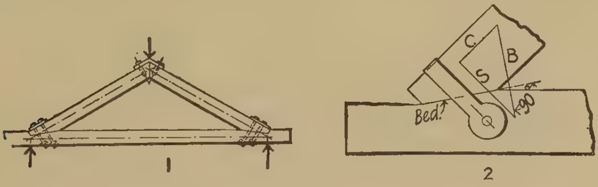
Plates III and IV give a part of the general and detail drawings of a roof truss for a first-class freight depot. The plans were originally prepared as a standard, but later they were superseded by special plans prepared for each location. One of the distinctive features consists in the adoption of cast-iron angle blocks with projecting lugs and pins, so that the braces are all cut square at each end, with a hole bored to receive the projecting cast pin. The end joint has double steps. The lower chord is built up of five 2- by 10-in. planks, arranged to break joints as indicated on Plate IV. It will be observed that all the joints are concentrated in the two panels adjacent to the center, without exceeding a length of 24 ft. for the planks. The splice bolts act as beams in transmitting the stresses from the end of one plank to the adjacent planks which continue past the joint. The bolster is cogged (Art. 32) over the wall plate and is continued beyond the connection of the angle brace, which is a housed mortise-and-tenon joint. The braces of the overhang on the outside have mortise-and-tenon joints without the housing.

The roof is arranged to dispense with common rafters. The purlins are small, in section like joists or rafters, and spaced close together, the sheathing being laid directly on the purlins. At the peak two purlins of increased depth are spiked together, and take bearing on top of the bent-plate washer.

Problem 70. Estimate the difference in cost for the alternative composition of the lower chord as given in the note on Plate III.

Art. 71. Tests of End Joints

Ten full-size tests to destruction of different kinds of end joints in roof trusses were made in the engineering laboratory of Massachusetts Institute of Technology, the detailed results of which, accompanied by dimensioned drawings and halftone illustrations of the manner of failure, were published in Technology Quarterly, September, 1897, and subsequently reprinted in pamphlet form. Figures 71*a*–*l* are reprinted from a review of these tests by F. E. Kidder in Engineering Record, vol. 42, page 464, November 17, 1900. Apart from the halftone illustrations, this article is the more valuable for reference, since the author gave the description copied from the records, and the results of his own computations of unit stresses in the connections, deductions, and conclusions. See also page 624 in the same volume for additional discussion on the subject.



Figs. 71*a* and *b*.—Tests of End Joints of Roof Trusses.

Figure 71*a* indicates the form in which the trusses were tested, the arrows indicating the points where the load was applied and the truss supported. The timber was hard pine in all cases.

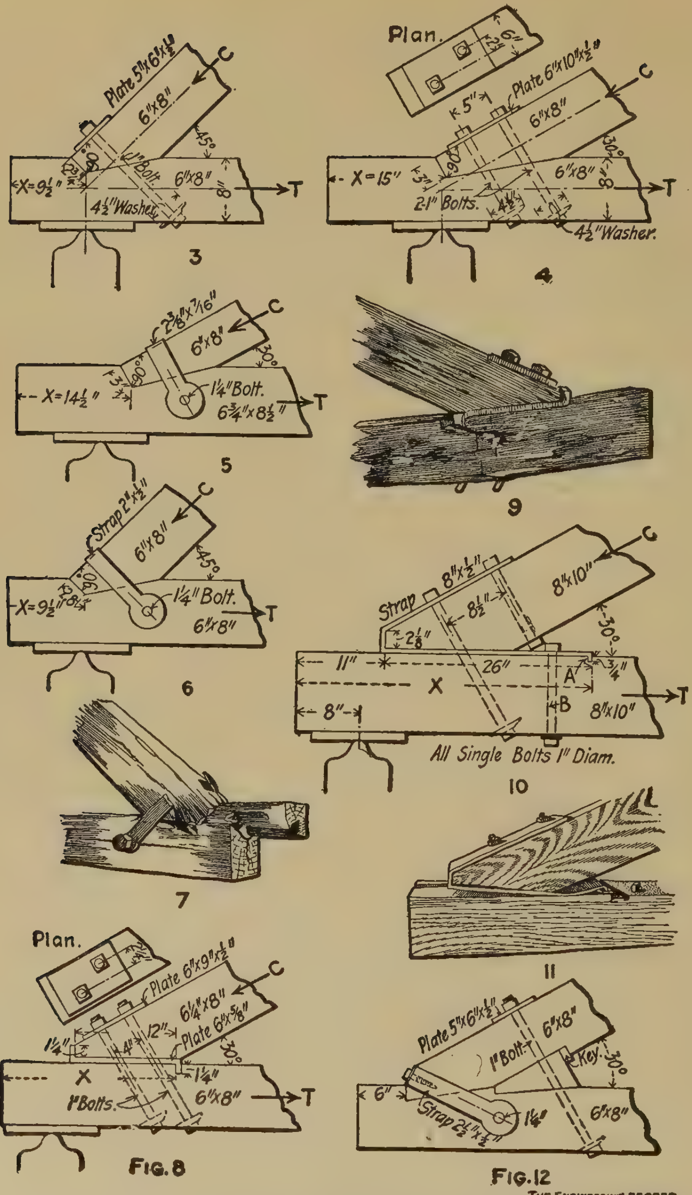


Plate 5"x6"x½"
6"x8"
C
45°
90°
X=9½"
1" Bolt.
6"x8"
8"
T
4½" Washer.
3
Plan.
6"
Plate 6"x10"x½"
5"
6"x8"
30°
X=15"
3"
90°
6"x8"
8"
2-1" Bolts.
4½
4½" Washer.
4
2⅜"x7/16"
6"x8"
30°
90°
X=14½"
3"
1¼" Bolt.
6¾"x8½"
5
9
Strap 2"x½"
6"x8"
45°
90°
X=9½"
1¼" Bolt.
6"x8"
6
Strap 8"x½"
8"x10"
8½"
2⅛"
30°
11"
26"
¾"
A
X
B
8"
8"x10"
All Single Bolts 1" Diam.
10
7
11
Plan.
Plate 6"x9"x½"
6¼"x8"
Plate 6"x⅝"
1¼"
4"
12"
30°
X
1¼"
1" Bolts.
6"x8"
FIG. 8
Plate 5"x6"x½"
1" Bolt
6"x8"
Key
30°
6"
Strap 2½"x½"
1¼"
6"x8"
FIG. 12
THE ENGINEERING RECORD.

FIGS. 71c–l.—Tests of End Joints of Roof Trusses.

The most important result obtained from this set of experiments is that the maximum resistance of the timber to horizontal shear beyond the step joint and that due to compression on the side of the upper chord timber by through bolts or straps do not occur simultaneously. In other words, the joint and fastenings, or two sets of fastenings, do not work in harmony. The lesson to be learned from the tests by designers is that the joint should be designed so as to depend either on one or the other element of resistance.

The disadvantage of the resistance of inclined bolts or straps to resist the horizontal component of a given stress in the upper chord is indicated in Fig. 71*b*; the stress in the bolt or strap must be considerably larger than the component force to be resisted, and since the bearing is applied on the side of the fiber, weak elements of the timber are brought into action, for the timber is weak in its resistance to compression on the side of the fibers, and yields considerably at the same time.

The two forms of shoe are shown to be superior to the step joints. This is largely due to insufficient provision for lengths of shearing surfaces; but herein lies one of the advantages of the shoe, since the shearing surface can be extended inside of the toe, for usually the distance outside of the toe is limited by other conditions of the structure. The shoe in Fig. 71*h* (No. 8) would have had considerably increased strength if the toe had been deeper and the shoe plate had been continuous with the upper bearing plate. The shoe in Fig. 71*j* would also have been materially stronger if a hook bolt had been used directly over the lug. As the resistance of the lug depends upon its strength in flexure, the strength of the plates in the two cases are to each other as 25 is to 16.

Nine tests of full-size wooden trusses with spans from 24 to 32 ft. were made under the direction of G. K. Scott Moncrieff, the results of which are given by him in an article in Journal of Royal Institute of British Architects, vol. 6, page 113. An abstract of this article was published in Engineering Record, vol. 39, page 284. The object of the experiments was " to ascertain whether trusses, as constructed according to the dimensions generally accepted in English practice, had any advantage, from a practical point of view, over other designs more strictly in accordance with theory and more economical of material." One detail of design deserving especial mention which was emphasized by these experiments relates to the importance of using cylindrical bearing blocks of hard wood when strap or U-bolts are employed. In the end joint of trusses especially designed for the tests the bolts were placed in a horizontal position, instead of the usual inclination.

Four tests of triangular trusses with end joints like those in Fig. 66*b* were made in the laboratory of the College of Civil Engineering of Cornell University in 1902 by Guy E. Long. The timbers were of longleaf yellow pine, having a compressive strength for short blocks of 7590 lb. per sq. in., this value being the mean of 12 tests. The wrought-iron shoes gave no indication in any test of yielding. The plates were $\frac{3}{4}$ in. thick, and the timbers were 6 by 6 in. in section, the upper chords having an inclination of 30 deg. The lugs engaged notches $1\frac{1}{2}$ in. deep. The effective span was 6 ft. $1\frac{1}{4}$ in., and the shearing length beyond each lug was $12\frac{1}{2}$ in.

Since the span was so short, a difference in the deflection of the lower chord could be observed when blocks were used to hold down the inner lugs, and afterwards bolts were substituted for the same purpose. The most important facts obtained by this investigation relate to the effect of secondary flexure due to the eccentric application on the lower chord of the horizontal component of the upper chord. A difference of $\frac{1}{2}$ in. each way in the location of the supports from the correct position could be detected by opposite deflections of the lower chord.

In one case the truss was loaded with the supports at the intersection of the center lines of the chords until shearing occurred between the lugs. The supports were than placed directly beneath the outer lugs, and when the load was reapplied the lower chord deflected upward and split the timber inward from the bottom of the notch at the inner lug. The supports were then placed at the correct position, which is intermediate between the other two, and in this condition the truss supported a greater load than before, notwithstanding the two previous fractures of the timber at the joint.

In another test the truss was subjected to a load nearly 1.25 times that for which it was designed without showing any signs of weakness, the correct position for the supports having been found previously by trial on the basis of deflection. The supports were then moved outward $3\frac{1}{2}$ in., and upon reloading the truss the timber sheared under a load of 76 percent of the previous one.

Problem 71. Compute the working strength of the lower chord in Fig. 71*h* (No. 8); first, with respect to the shearing resistance of the timber; second, with respect to the compressive resistance of the toe; and third, with regard to the strength of the lug. Let the following working unit stresses be assumed: Compression on the ends of the fibers, 1500; shear parallel to the fiber, 150; stress in the outer fiber of the lug, 20,000 lb. per sq. in.

Art. 72. Details of Roof Trusses

Figure 72*a* shows some characteristic details of a roof truss designed by Henry Goldmark for the freight-car shops of the Canadian Pacific Railway at Montreal. The truss has a length out to out of 102 ft. 2 in., supported by a column of steel at the middle. The depths over all at the ends and center are 7 ft. 3 in.

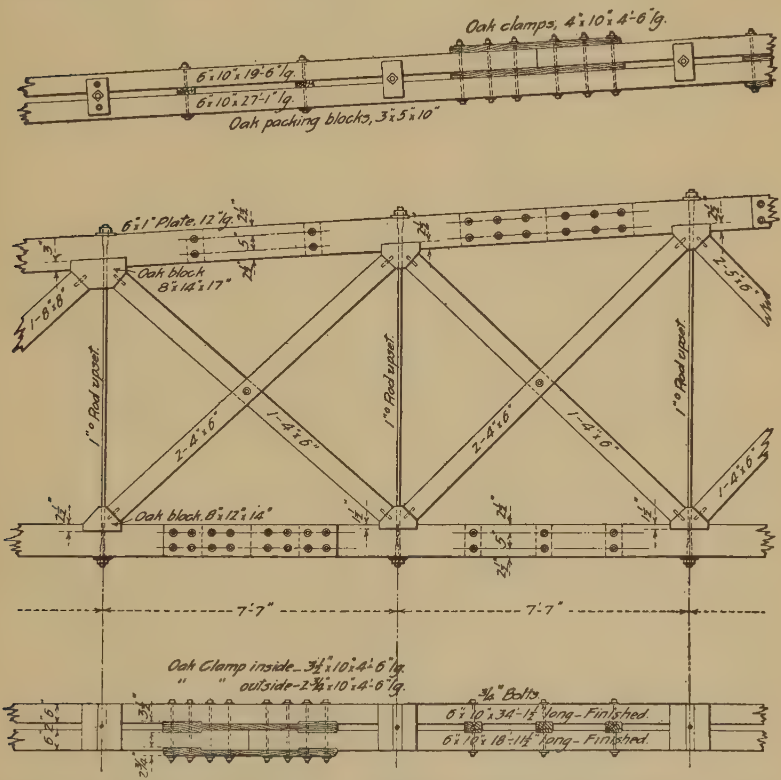

Oak clamps, 4"x10"x4'-6" lg.
6"x10"x19'-6" lg.
6"x10"x27'-1" lg.
Oak packing blocks, 3"x5"x10"
6"x1" Plate, 12" lg.
Oak block 8"x14"x17"
1'-8"x8"
1" Rod upset.
2'-4"x6"
1'-4"x6"
2'-5"x6"
Oak block, 8"x12"x14"
7'-7"
7'-7"
Oak Clamp inside-3½"x10"x4'-6" lg.
" " outside-2¾"x10"x4'-6" lg.
¾" Bolts.
6"x10"x34'-1½" long-Finished.
6"x10"x18'-11½" long-Finished.
6
6
3½
2¾

Fig. 72*a*.—Part of Roof Truss for Freight-car Strops of Canadian Pacific Railway at Montreal.

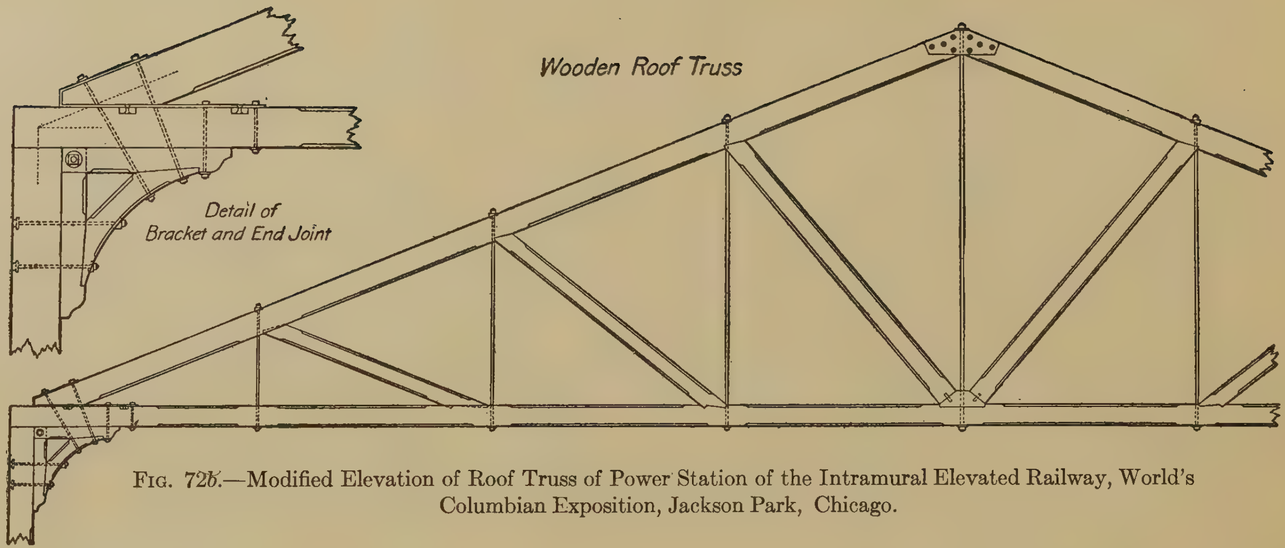


FIG. 72b.—Modified Elevation of Roof Truss of Power Station of the Intramural Elevated Railway, World's Columbian Exposition, Jackson Park, Chicago.

The span is 77 feet 9 inches, the rise is one-fifth of the span, and the trusses are spaced 15 feet between centers. The upper chord is composed of 5 planks 2 by 12 inches, which break joints and are bolted together with ½-inch bolts every 2 feet. The lower chord is built up in a similar manner of 2- by 10-inch planks. The braces are single sticks 8 by 8, 8 by 10, and 10 by 10 inches in section. The rods are ¾, ¾, 1⅛ and 1¾ inches in diameter. The wrought-iron splice plates at the peak are fastened with ½-inch bolts. The shoe plate is 10 by ⅝-inch in section, and the inclined bolts are 1 inch in diameter. The truss is framed with a camber of 1½ inch in the lower chord.

and 10 ft. 9 in., respectively. The trusses are spaced 20 ft. center to center. The illustration shows the splices of both upper and lower chords, as well as the spacing blocks between the timbers. Those in the lower chord are notched into both timbers to aid in equalizing the stresses between them. The braces are held in position by dowels against the oak angle blocks. For additional illustrations and their description see an article on Canadian Pacific Railway Shops at Montreal; Part III.—Structural Features in the Blacksmith, Machine, Wood-working, and Other Shops, of Engineering Record, vol. 49, page 417.

At the end joint of the truss, the inclined brace has a single step engaging the lower chord, and a head block is placed outside, bearing against the inclined surface of the brace. A bolster is keyed to the bottom of the lower chord, and a long inclined bolt takes the thrust of the head block and reacts against the inner end of the bolster. The head block and bolster are also united by two vertical bolts passing between the lower chord timbers.

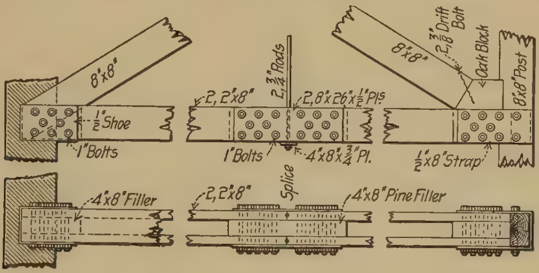


Fig. 72c.

Figures 72*c*, *d*, and *e* illustrate some details of roof trusses which were originally contributed to Engineering News in response to a request for criticism of a partial plan submitted by a correspondent. They are reprinted here for the purpose of indicating certain defects in design which are prevalent in practice. The feature which first attracts attention in Fig. 72*c* is the plain fish-plate joint in the lower chord, in which three rows of bolts are placed in an 8-in. fish plate. Apparently no allowance has been made for the tendency of round bolts acting as beams to split the timber. The longitudinal distance between the staggered bolts of adjacent rows is so short that nothing seems to be gained by staggering them.

At the left end joint the shoe plate is bent twice at right angles, and is supposed to resist bearing over the middle half of its horizontal end width. This condition involves flexure, for which a half-inch plate is inadequate. The joint would be improved by rounding the end bearing so as to reduce the flexure in the plate. Some of the bolts are too close to the edge of the upper chord timber and of the filler below it.

The design in Fig. 72*d* was offered as an improvement upon that in Figs. 72*c* and *e*. Its element of weakness consists in the apparent failure to consider the pressure on the side of the fibers due to the moment of rotation in the angles. If each horizontal rod is properly designed, so far as the net section area due to direct tension is concerned, what probable stresses in the outer fiber of the net section may be caused by the uneven bearing of the nut on the angle?

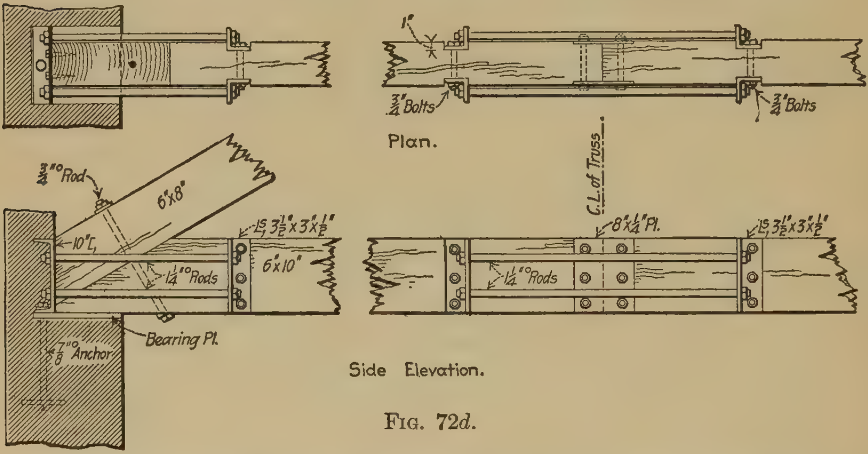


FIG. 72*d*.

In Fig. 72*e* attention is at once directed to the angle brace which is extended to the upper chord of the truss, and therefore introduces ambiguity into the stresses for a number of truss members. The truss becomes a statically indeterminate structure. The keys in the fish plate are so short longitudinally with respect to the chord, that if subjected to the working strength of the other parts of the joint, they will either rotate or approach so near to this condition as to lack a fair degree of security. The keys receive bearing on the sides of their fibers, thus lowering the allowable unit-stress in compression. Since they also tend to shear in a plane parallel to the fibers, but in a direction transverse to the fibers, a physical property of wood is involved, concerning which no published results of tests are available.

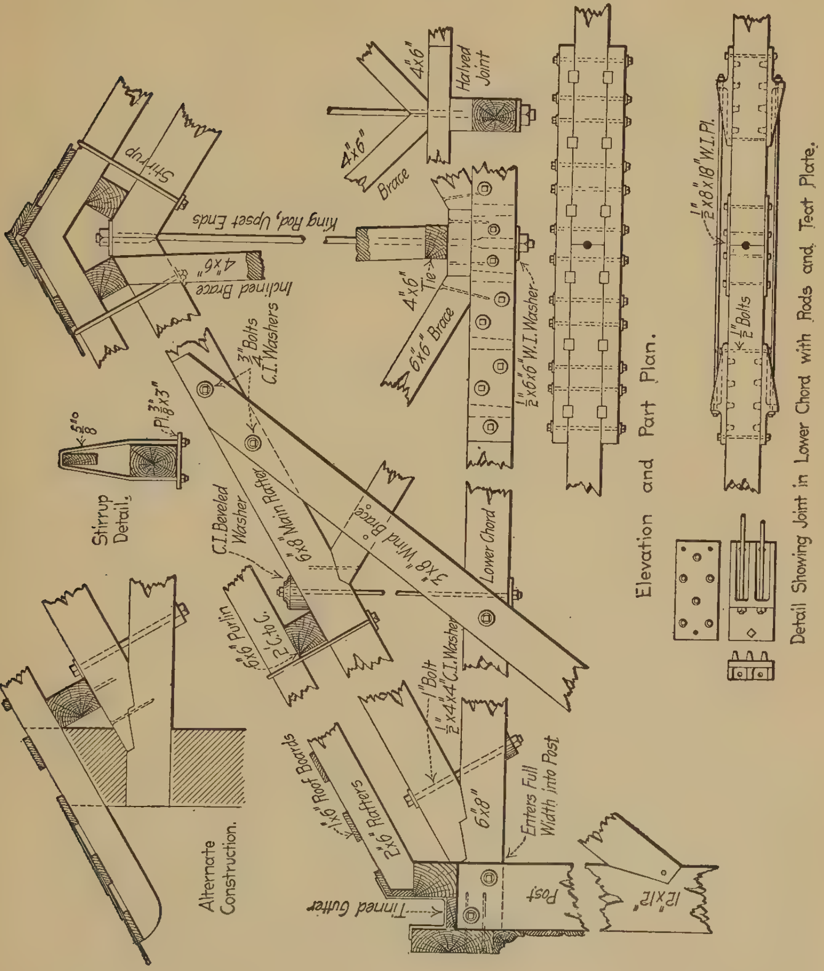

Stirrup
King Rod, Upset Ends
4"x6" Inclined Brace
3/4" Bolts C.I. Washers
Pl. 3/8"x3"
5/8"o
Stirrup Detail.
4"x6"
Halved Joint
4"x6" Brace
4"x6" Tie
6"x6" Brace
1/2"x6"x6" W.I. Washer
Elevation and Part Plan.
1/2"x8"x18" W.I. Pl.
1/2" Bolts
Detail Showing Joint in Lower Chord with Rods and Teat Plate.
C.I. Beveled Washer
6"x8" Main Rafter
3"x8" Wind Brace
Lower Chord
6"x6" Purlin 2' C. to C.
1" Bolt
1/2"x4"x4" C.I. Washer
1"x6" Roof Boards
2"x6" Rafters
6"x8"
Enters Full Width into Post
Post
12"x12"
Tinned Gutter
Alternate Construction.

FIG. 72e.

In regard to the splice with cast-iron plates connected by rods, it may be questioned whether the conical projections furnish sufficient bearing area according to the principles given in Art. 24. Where a number of bearings are to act in harmony, the holes should be bored with the aid of a templet which corresponds exactly with the casting. In the end joint the second step would be more effective if the longer face of the triangular notch coincided with the lower surface of the upper chord timber. The proposed modification either increases the length of shearing surface between the notches, or places the two shearing surfaces at different heights, depending upon the relative depths of bearing adopted (Art. 35). The sections of the purlins are not well proportioned to develop their strength as beams. The so-called stirrup used to hold down the rafter is an expensive fastening, for which spikes and a wooden block may be substituted.

Figure 72*f* gives an outline diagram of one of the main roof trusses of the Forestry building, and Fig. 72*g* shows the kind of

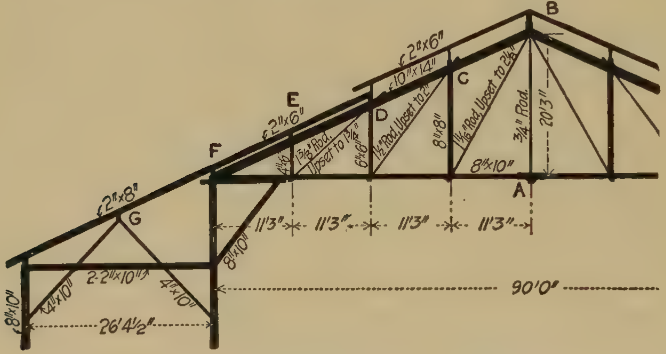


Fig. 72*f*.—Outline of Main Truss, Forestry Building.

details which are apparently necesary when an attempt is made to construct chiefly in wood a type of roof truss which is primarily adapted for wrought-iron or steel. An unusually long cast-iron shoe receives the bearing of the upper chord, and the casting at the peak is very elaborate. As only a single diagonal rod is used on each side of that joint, there is a rotating moment developed which produces secondary bending stresses in the chord timbers. The three beveled washers shown are of the type illustrated in Fig. 2*k*, except that they are single instead of double. The splice in the lower chord is a combination of the tabled and plain fish-plate joint (see Art. 31). It should be especially noted that the trussed purlins are placed with their sides horizontal and vertical (see Art. 60). The lower chord and bolster are connected to the supporting post by a plaster joint. The details also show the upper end of the knee brace connecting the roof truss and post, and the relation to its joint with the lower chord to the adjacent panel point of the truss.

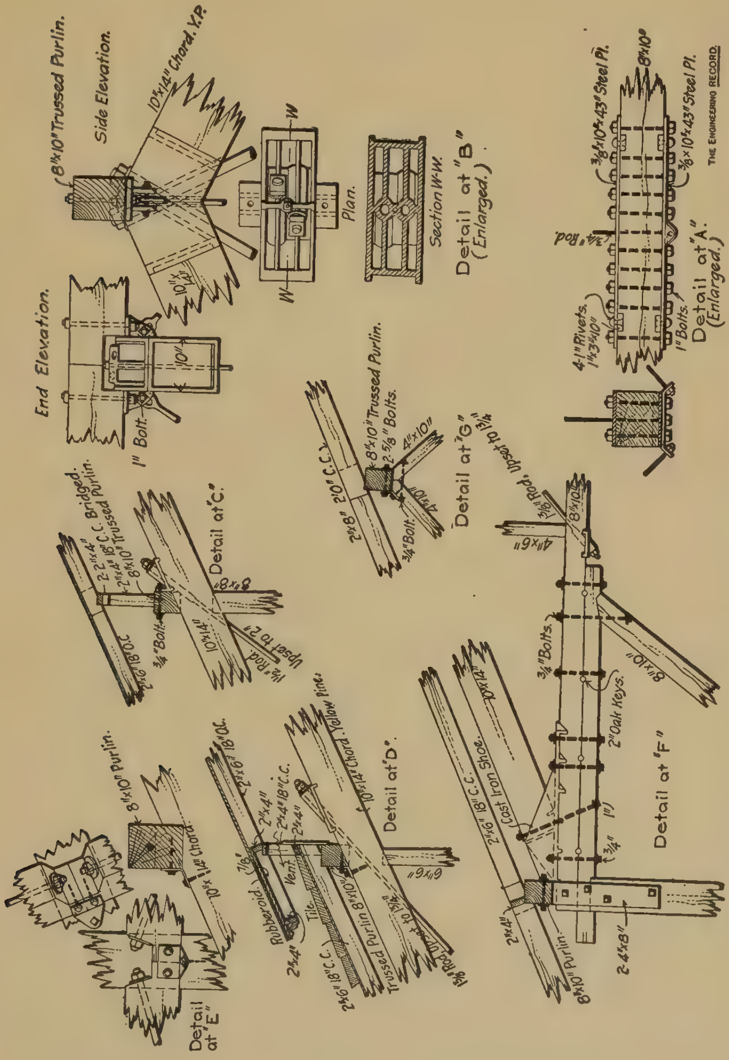

8"x10" Trussed Purlin.
Side Elevation.
10"x14" Chord, Y.P.
10"x14"
W
W
Plan.
Section W-W.
Detail at "B" (Enlarged.)
End Elevation.
10"
1" Bolt.
3/8"x10"x43" Steel Pl.
8"x10"
3/8"x10"x43" Steel Pl.
3/4" Rod.
4-1" Rivets. 1"x3"x10"
1" Bolts.
Detail at "A." (Enlarged.)
2-2"x4" 18" C.C. Bridged.
2"x4" 18" C.C.
8"x10" Trussed Purlin.
3/4" Bolt.
10"x14"
2"x6" 18" O.C.
8"x8
1 1/4" Rod, Upset to 2"
Detail at "C."
2"x8" 2'0" C.C.
8"x10" Trussed Purlin.
2-5/8" Bolts.
4"x10"
3/4" Bolt.
10"x14"
Detail at "G"
3/4" Rod, Upset to 1 3/4
4"x6"
8"x10
3/4" Bolts.
2" Oak Keys.
8"x10"
1"
3/4"
Detail at "F"
8"x10" Purlin.
10"x14" Chord.
Detail at "E"
2"x6" 18" O.C.
2"x4"
2"x4" 18" C.C.
2"x4"
Vent.
Tile.
Rubberoid.
2"x4"
2"x6" 18" C.C.
Trussed Purlin 8"x10"
10"x14" Chord. Yellow Pine.
Detail at "D".
6"x6"
1 1/2" Rod, Upset to 1 3/4"
2"x6" 18" C.C.
10"x14"
Cast Iron Shoe.
2"x4"
8"x10" Purlin.
2-4"x8"

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Fig. 72g.—Details of Roof Truss of Forestry Building, Pan-American Exposition.

ART. 73. EXAMPLES FROM PRACTICE

Figure 73*a* gives the elevation of the half span of one of the main roof trusses for the Mess Hall and Kitchen of the War College and Engineer Post at Washington, D. C. It was designed by Ewald Schmitt. The effective span is 43 ft. 8 in., the depth center to center of chords is 9 ft. 7 in., and the trusses are spaced 11 ft. 11 in. between centers. The sheathing of plank is supported directly by the trusses, and the slate roofing is laid upon that. The trusses were designed for a total load of 60 lb. per sq. ft., one-half of which is an allowance for both snow and wind loads. The details are clearly shown in the illustration.

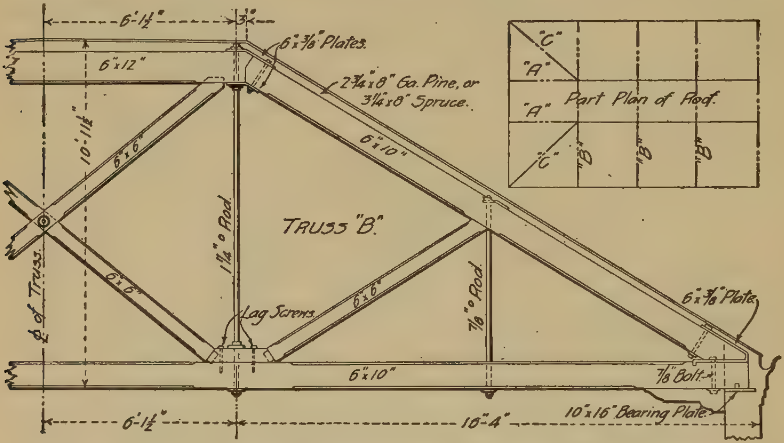

6'-1½"
3"
6"x⅜" Plates.
6"x12"
2¾"x8" Ga. Pine, or
3¼"x8" Spruce.
"C"
"A"
"A"
Part Plan of Roof.
"C"
"B"
"B"
"B"
10'-11½"
6"x6"
6"x10"
1¼"⌀ Rod.
TRUSS "B."
℄ of Truss.
6"x6"
Lag Screws.
6"x6"
⅞"⌀ Rod.
6"x⅜" Plate.
6"x10"
⅞" Bolt.
10"x16" Bearing Plate.
6'-1½"
16'-4"

FIG. 73*a*.—Roof Truss for Mess Hall and Kitchen of War College and Engineer Post, Washington, D. C.

Figure 73*b* shows the elevation of one-half span of a roof truss and a half cross-section of a freight warehouse at Atlanta, Ga., of the Central of Georgia Railway. The roof trusses are very simple in design, consisting entirely of wood and connected at the panel points by bolts. Since the braces consist of single sticks and are held between the chord timbers in the manner shown, the center lines of truss members do not intersect in a point at the panel points. This involves secondary bending moments which are taken into account in designing the section areas of the chords. The span, depth, and composition of the truss members are given on the drawing.

Figure 73*c* gives the elevation of the half span of one of the main roof trusses of the north wing of Goldwin Smith Hall at Cornell University. This portion of the building was originally built for the Dairy Department of the College of Agriculture. The span and rise of the truss are given on the drawing. The trusses are spaced 8 ft. apart, and directly support the heavy plank sheathing which was laid diagonally over the larger portion of the roof to secure more rigid bracing. The roof covering consists of a heavy glazed interlocking tile. The details of the pin-connected joint are shown on an enlarged scale as well as those of the bent bearing plate at the end joint of the truss. The strut enters the socket of a casting at the lower end which takes bearing on the pin.

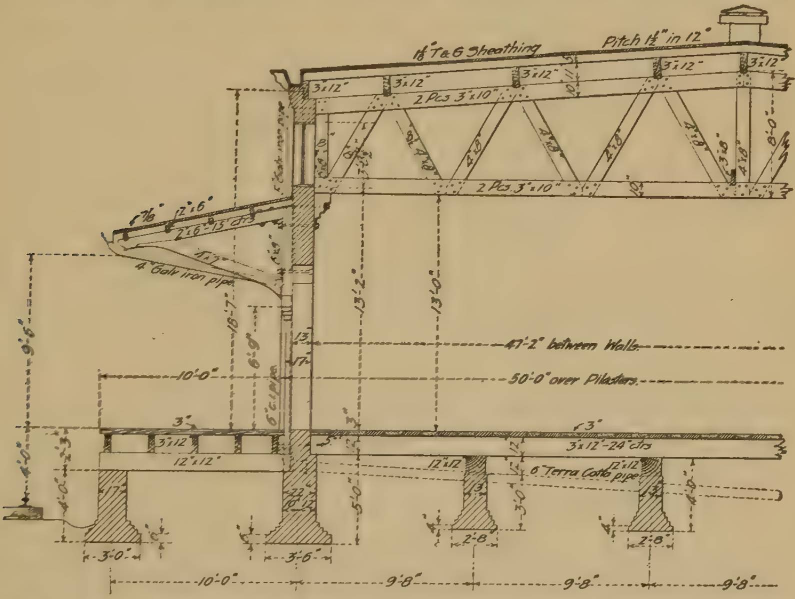

1⅝" T&G Sheathing
Pitch 1½" in 12"
3"x12"
3"x12"
3"x12"
3"x12"
3"x12"
2 Pcs 3"x10"
2 Pcs 3"x10"
8'-0"
2"x6"
2"x6"-13" c/rs
4" Galv iron pipe
18'-7"
6'-9"
13'-2"
13'-0"
47'-2" between Walls
50'-0" over Pilasters
10'-0"
9'-6"
3"
3"
3"x12"
12"x12"
12"x12"
3"x12"-24" c/rs
6" Terra Cotta pipe
12"x12"
4'-0"
5'-0"
3'-0"
3'-0"
3'-6"
2'-8"
2'-8"
10'-0"
9'-8"
9'-8"
9'-8"

FIG. 73*b*.—Cross-section of Freight Warehouse of Central of Georgia Railway at Atlanta, Ga.

The upper drawing on Plate V shows a roof truss which is built mainly of yellow pine, steel being used for a few of the web tension members and for splices and connection bolts. The trusses divide the roof into three bays of about 22 ft. and two of 27 ft., and are proportioned for the standard roof loads prescribed by the building laws. The trusses supporting the longer bays differ from that

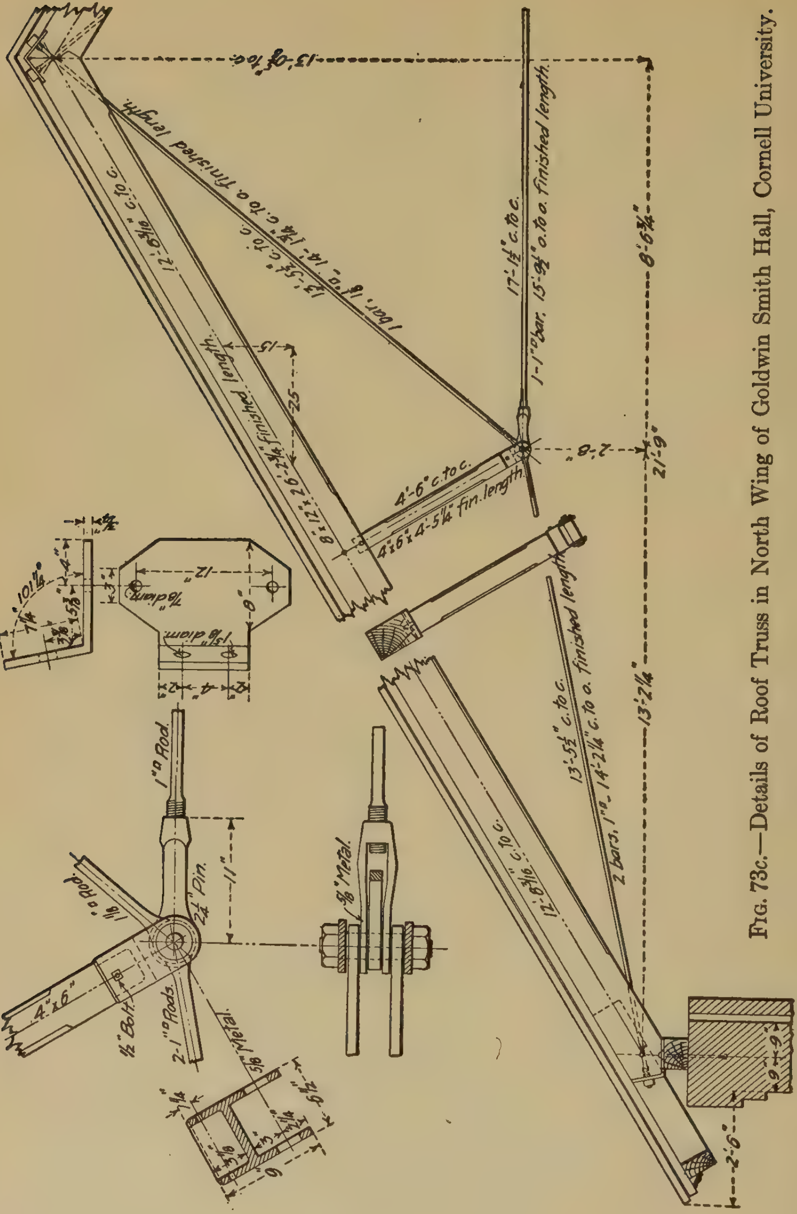
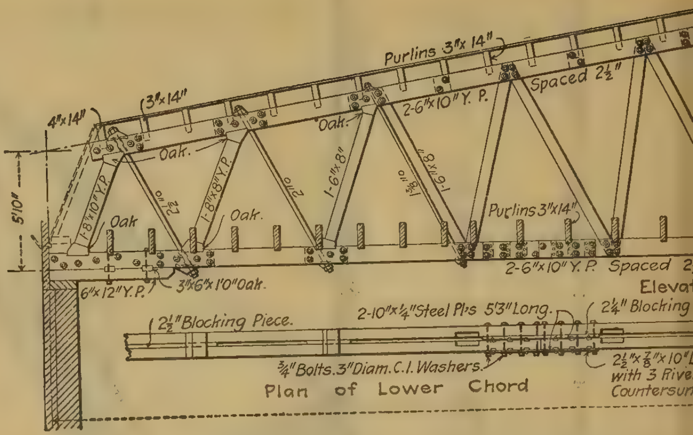
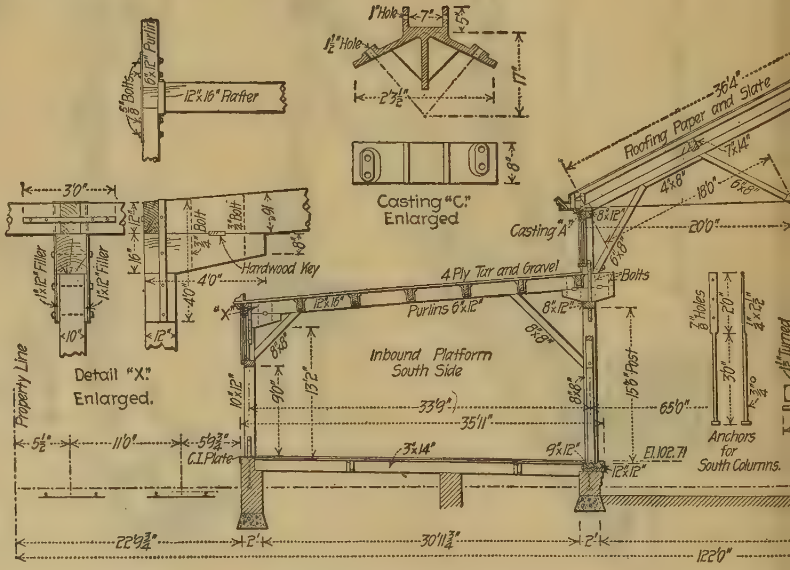

13'-0⅝" to c.
1 bar, 1⅛" o. 14'-1¾" c. to c.
13'-5½" c. to c.
finished length
12'-6 3/16" c. to c.
8"x12" 26'-2¾" finished length
15
25
17'-1½" c. to c.
1-1" bar. 15'-9¾" o. to o. finished length
8'-6¾"
2'-8"
21'-9"
4'-6" c. to c.
4"x6" 4'-5¼" fin. length
13'-2¼"
13'-5½" c. to c.
2 bars, 1" 14'-2¼" c. to o. finished length
12'-6 3/16" c. to c.
2'-6"
9
9
12
8
⅞" diam.
1 5/16" diam.
2
4
2
101½
7¼
4
3
1 5/8
3/8
1" Rod.
11"
2½" Pin.
⅝" Metal.
1 1/16" Rod
4"x6"
½" Bolt.
2-1" Rods.
5/16" Metal.
6½
9
3
¼
¼

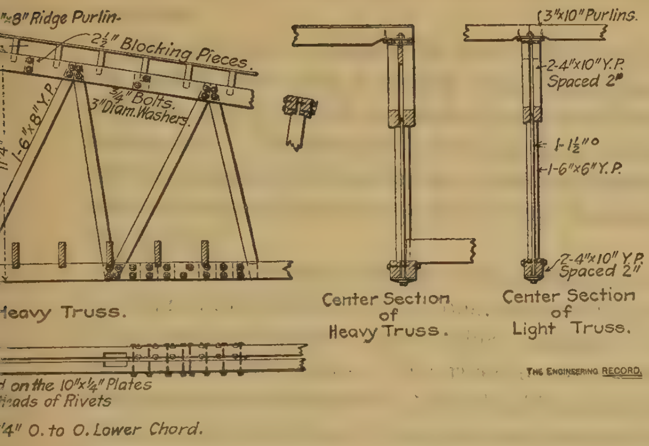
FIG. 73c.—Details of Roof Truss in North Wing of Goldwin Smith Hall, Cornell University.


Purlins 3"x 14"
Spaced 2½"
2-6"x10" Y. P.
(3"x14"
4"x14"
Oak.
Oak.
1-8"x10" Y.P.
Oak
2½"⌀
1-8"x8" Y.P.
Oak.
2½"⌀
1-6"x8"
1⅜"⌀
1-6"x8"
Purlins 3"x14"
5'10"
6"x12" Y.P.
3"x6"x1'0" Oak.
2-6"x10" Y.P. Spaced 2
Eleva
2½" Blocking Piece.
2-10"x¼" Steel Pl's 5'3" Long.
2¼" Blocking
¾" Bolts. 3" Diam. C.I. Washers.
2½"x⅞"x10" L
with 3 Rive
Countersun
Plan of Lower Chord

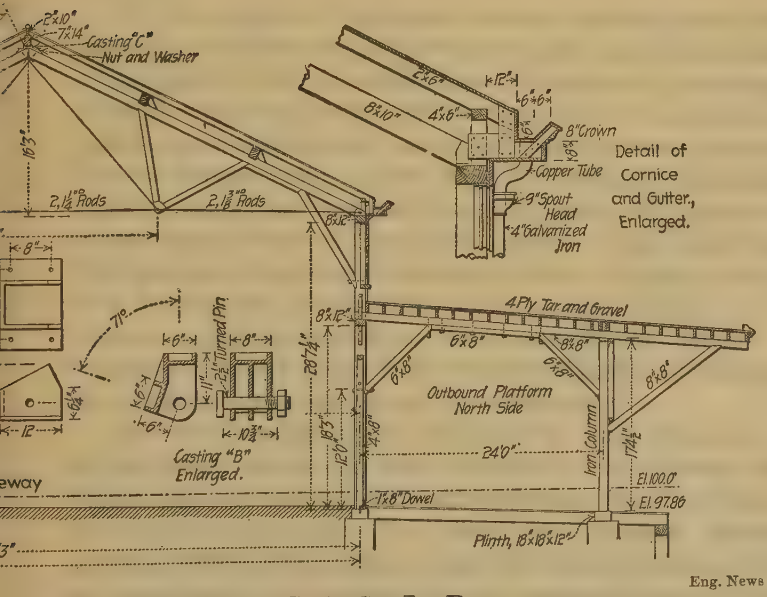
Details of a Wooden Roof


1" Hole
7"
5"
1½" Hole
17"
2'3½"
8"
Casting "C."
Enlarged
5/8" Bolts
6"x12" Purlin
12"x16" Rafter
3'0"
1"x12" Filler
1"x12" Filler
10"
Detail "X."
Enlarged.
12"
16"
4'0"
12"
Bolt
¾" Bolt
16"
8"
Hardwood Key
4'0"
Roofing Paper and Slate
36'4"
7"x14"
4"x8"
18'0"
6"x8"
20'0"
Casting "A"
8"x12"
6"x8"
4 Ply Tar and Gravel
Bolts
"X"
12"x16"
Purlins 6"x12"
8"x12"
8"x8"
8"x8"
Inbound Platform
South Side
10"x12"
9'0"
13'2"
33'9"
35'11"
8"x8"
15'6" Post
65'0"
7/8" Holes
2'0"
3'0"
¼"x2½"
3/4"⌀
1½" Turned
Anchors
for
South Columns.
Property Line
5½"
11'0"
5'9¾"
C.I. Plate
3"x14"
9"x12"
El. 102.71
12"x12"
22'9¾"
2'
30'11¾"
2'
122'0"

Cross-Section of Central Ave. Freight


"×8" Ridge Purlin
2½" Blocking Pieces.
¾" Bolts.
3" Diam. Washers
1-6"×8" Y.P.
11'4
leavy Truss.
d on the 10"×¼" Plates
Heads of Rivets
'4" O. to O. Lower Chord.
3"×10" Purlins.
2-4"×10" Y.P. Spaced 2"
1-1½"○
1-6"×6" Y.P.
2-4"×10" Y.P. Spaced 2"
Center Section of Heavy Truss.
Center Section of Light Truss.
THE ENGINEERING RECORD.

Sterling Place, Brooklyn.


2"×10"
7"×14"
Casting "C"
Nut and Washer
16'3"
2, 1¼"○ Rods
2, 1⅛"○ Rods
8"×12"
8'
71°
6"
8"
Turned Pin
11"
2½"
6¼"
6"
10¾"
12
Casting "B" Enlarged.
eway
8"×12"
28'7¼"
18'3"
12'0"
4"×8"
1"×8" Dowel
2"×6"
8"×10"
4"×6"
12"
6"×6"
6"
8" Crown
8¾"
Copper Tube
9" Spout Head
4" Galvanized Iron
Detail of Cornice and Gutter, Enlarged.
4 Ply Tar and Gravel
6"×8"
8"×8"
6"×8"
6"×8"
8"×8"
Outbound Platform North Side
24'0"
Iron Column
17'4½"
El. 100.0'
El. 97.86
Plinth, 18"×18"×12"
3"
Eng. News

at Cincinnati ; C. C. C. & St. L. Ry.

shown only in having slightly larger cross-sections in some of the members.

The top chord timbers all reach from the end of the truss to the center and make a simple miter-joint there under a ribbed cast-iron saddle, which is beveled to fit over the ridge but is not bolted to it. The vertical center rod takes bearing on this casting, and its upper end is received in a hole mortised in the ridge purlin, which is secured there by a wood screw through the top flange of the saddle.

The bottom chord is made of four sticks, two of them 43 ft. 10 in. long, and two 23 ft. 6 in. long, arranged to break joints 10 ft. 2 in. from the center. Each splice consists of a tabled fish-plate joint with steel plates. The rivet heads on the outside of the plates are not countersunk. A wooden filler is put between the chord pieces at the splices and there are two rows of bolts through both the continuous timber and the splice, arranged so that there is a pair of bolts just beyond each bearing strip counting from the splice. It should be noted that in practice the bolts are not placed in their proper position to resist the rotating moment as explained in Art. 22.

In the middle of the truss, the web members have shoulders bearing on shallow seats mortised in the inner horizontal faces of the chords, over the whole area of intersection, and they have beveled tenons extending beyond the seats to the outside of the chords and abutting with the adjacent web member, being secured by two through bolts. At the ends of the trusses, the larger compressive stresses are resisted by inclined posts with square ends set on oak angle blocks notched $1\frac{3}{4}$ in. into the chords across the face. The corresponding tensile stresses are taken by single round screw rods having nuts bearing on cast-iron angle blocks or beveled washers engaging notches on the outside of the chords. All of them are the width of the chord, but their lengths and the depths of the toe pieces are proportioned to the stresses.

The end bearings are in bolsters 6 ft. long, which are keyed to the lower chords and were intended to be stiffened by knee braces, but the latter were omitted because they interfered with the required clearance. The design was especially intended to secure an efficient disposition of material, simple framing, economical construction, to develop the full strength of the members in the connections and splices and to have the separate pieces so secured that they will not be loosened nor permit deformation to result from shrinkage of the timber. The design was made by G. H. Chamberlain and Bernt Berger, the architect and consulting engineer, respectively.

The lower drawing on Plate V shows the cross-section of the Central Avenue freight station at Cincinnati, Ohio, of the Cleveland, Cincinnati, Chicago, and St. Louis Railway, built to replace an old structure destroyed by fire. Most of the general dimensions and a number of details are given on the drawing, the latter to an enlarged scale. The main columns of the building are 21 ft. apart longitudinally and carry wall plates 8 by 12 in. for the support of the platform roofs, and above these the column are carried up to support the wall plates of the central roof, the trusses of which are spaced 21 ft. between centers. The load is not carried entirely on the wall plates but is transferred by struts and braces to bearing points on the columns.

The combination roof truss has all its tension members composed of square bars. At the intermediate panel points of the lower chord the rods are looped over $2\frac{1}{2}$-in. turned pins, and at the end panel points over $1\frac{1}{2}$-in. pins. The upper ends of the inclined tension members take bearing on the casting at the peak by nuts and washers. The details of the three castings are shown.

The roof over the inbound platform has 12- by 16-in. rafter beams following the slope of the tar and gravel roof, which rest on bolsters supported by the main posts and by smaller posts along the edge of the platform. Between the beams are purlins supported by double stirrups, on which the $2\frac{1}{2}$-in. sheathing is laid. The roof over the outbound platform has rafter beams 8 by 8 in. in section, supported on the wall plates over the line of main columns and on iron columns on the platform, and they extend 7 ft. beyond to support the overhanging roof. Small purlins spaced close together rest on top of the beams, and in turn support $\frac{7}{8}$-in. sheathing. There is a system of angle braces throughout, and special attention is called to the use of a straining beam under the rafter beams of the outbound platform, and to the straining beam between the angle brace and the nearest strut of the roof truss. The illustration and data regarding this roof truss are taken from an article in Engineering News, vol. 46, page 16.

Art. 74. References to Engineering Literature

The following references relate only to roof trusses. No attempt is made to refer to all the engineering periodicals published in this country, nor to include every article on the subject to be found in the periodicals selected. The aim has been merely to give a sufficient number of selected references, to enable the student or young engineer to notice the types of roof trusses in use in this country and to study the details employed in various designs. Card catalogues of references, arranged in accordance with a definite system of classification, form a part of the equipment of first-class offices of engineers and architects.

WOODEN AND COMBINATION ROOF TRUSSES

Modern Industrial Plant.—Eng. Rec., v. 23, p. 356, May 2, 1891.

Tacoma Shops of the Northern Pacific Railroad; Combination Trusses of Large Span.—Eng. News, v. 27, p. 170 (inset), Feb. 20, 1892.

World's Columbian Exposition; Transportation Building.—Eng. News, v. 28, p. 605 (inset), Dec. 29, 1892.

Buildings and Structures of American Railroads, by Walter G. Berg, 1892.

World's Columbian Exposition; Electricity Building.—Eng. News, v. 29, p. 434 (inset), May 11, 1893.

Sanger Hall, Philadelphia; Design and Construction of a Large Temporary Auditorium.—Eng. Rec., v. 35, p. 120, Jan. 9, 1897.

New Freight Station at Columbus, Ohio, for the Pittsburgh, Cincinnati, Chicago, & St. Louis Ry.—Eng. News, v. 37, p. 66 (inset), Feb. 4, 1897.

New Boston & Maine Shops at Concord.—R. R. Gaz., v. 30, p. 76 (inset), Feb. 4, 1898.

New Foundry of the Semi-Steel Co.—Eng. News, v. 39, p. 405, June 23, 1898.

Scissors Truss, by F. E. Kidder.—Eng. Rec., v. 40, p. 440, Oct. 7, 1899; by W. C. West, v. 40, p. 488, Oct. 21, 1899; by F. E. Kidder, v. 40, p. 710, Dec. 23, 1899.

Special School House Roof Truss.—Eng. Rec., v. 42, p. 257, Sept. 15, 1900.

Ethnology Building at the Pan-American Exposition.—Eng. Rec., v. 44, p. 226, Sept. 7, 1901.

East Orange Town Hall Roof.—Eng. Rec., v. 44, p. 403, Oct. 26, 1901.

Memphis Shops of the Illinois Central.—R. R. Gaz., v. 34, p. 792, Oct. 17, 1902.

Bi-Centennial Memorial Building of Yale University.—Eng. Rec., v. 47, p. 604, June 6, 1903.

Details of a Large Timber Roof of a Public Shelter in Van Cortlandt Park, New York.—Eng. Rec., v. 49, p. 259, Feb. 27, 1904.

New B. & O. Freight House at Columbus.—R. R. Gaz., v. 38, p. 403, April 28, 1905.

New Freight Station at Cincinnati, O.; Cincinnati Southern Ry.—Eng. News, v. 54, p. 593, Dec. 7, 1905.

Saw-tooth Roofs for Factories, by Knight C. Richmond.—Eng. News, v. 56, p. 627, Dec. 13, 1906.

170-Ft. Wooden Roof Truss.—Eng. Rec., v. 55, p. 348, March 16, 1907.

Long Span Timber Roof Truss, by J. C. Lathrop.—Eng. News, v. 59, p. 628, June 11, 1908.

Standard Boiler House Design of the Oliver Iron Mining Co., by A. M. Gow.—Eng. News, v. 60, p. 294, Sept. 17, 1908.

Large Concrete and Timber Coal Pocket.—Eng. Rec., v. 59, p. 603, May 8, 1909.

Timber-arch Truss Roof with Steel Gusset Plates, by H. W. Sheley.—Eng. News-Rec., v. 80, p. 595, March 28, 1918.

Details of Heavy Timber Framing.—Eng. News-Rec., v. 84, p. 1016, May 20, 1920.

CHAPTER V

EXAMPLES OF FRAMING IN PRACTICE

ART. 75. SLOW-BURNING CONSTRUCTION

SLOW-BURNING or mill construction, as defined by Edward Atkinson, its most noted exponent, consists in so disposing the timber and plank in heavy solid masses as to expose the least number of corners or ignitable projections to fire, to the end also that when fire occurs it may be most readily reached by water from sprinklers or hose. This disposition consists in separating every floor from every other floor by incombustible stops, by automatic hatchways, by encasing stairways and belts either in brick or with incombustible partitions, so that a fire would be retarded in passing from floor to floor to the utmost that is consistent with the use of wood or any material in construction that is not absolutely fireproof. It also consists in guarding with fire-retardent material the ceilings over all specially hazardous stock or processes and in so constructing the mill, workshop, or warehouse that fire shall pass as slowly as possible from one part of the building to another, and furthermore in providing all suitable safeguards against fire.

A valuable illustrated description of this method of construction, and one containing general directions for designers, is published as Report 5 of the Insurance Engineering Experiment Station under the direction of the Boston Manufacturers Mutual Fire Insurance Company. It contains nine folding plates of plans which illustrate good practice and gives directions regarding details based upon extensive experience. The pamphlet also contains a table of weights of merchandise giving the approximate dimensions of the packages, their cubic contents, and floor spaces required, as well as the weights per square foot and per cubic foot.

The standard mill, usually having brick walls, is planned with heavy beams from 8 to 11 ft. between centers, and supported by posts composed of single sticks of square cross-section. In determining the size of beams, the three elements of weight, deflection and vibration are considered. A heavy plank floor, grooved and splined, is laid on the beams and on this is placed a single thickness of hard, close-grained floor boards with two layers of resin-sized paper between. In the best mills a board floor is laid diagonally or at right angles to the planks, and over that a top floor of birch or maple is laid lengthwise. This intermediate floor gives great resistance to lateral vibration, and can be ordinarily of the cheapest

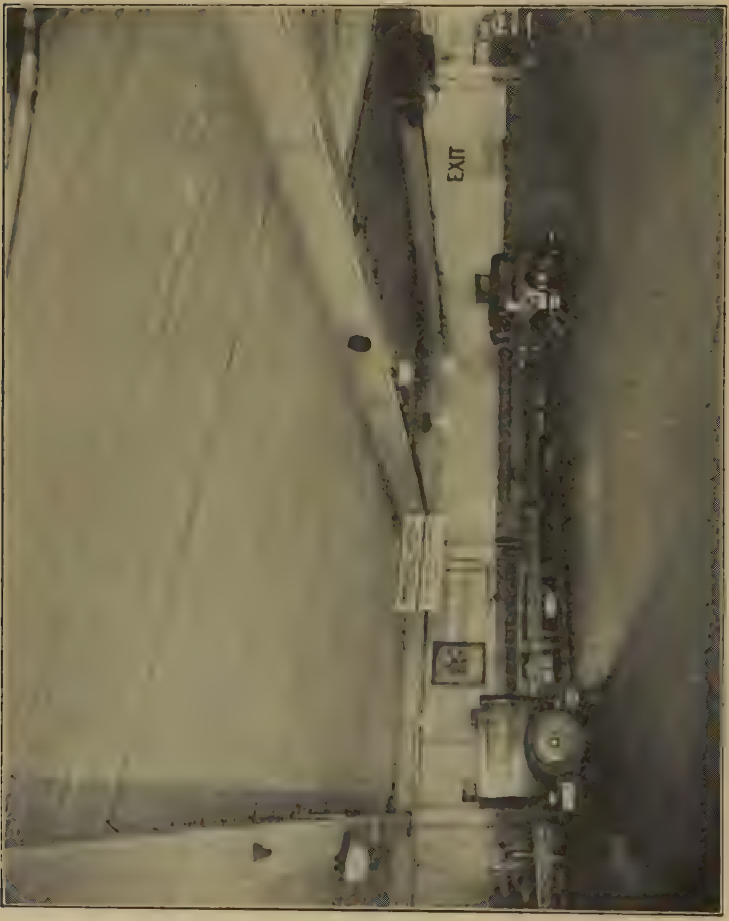


FIG. 75*a*.—Mill Construction with Laminated Floor.

lumber that is obtainable of practically uniform thickness, and is well worth the additional cost.

In some cases the flooring consists of 2-in. joists on edge spiked together closely side by side, the thickness of the floor varying with the loads and span from 5 to 8 in. or more. This is called mill construction with laminated floor and is illustrated in Fig. 75*a*.

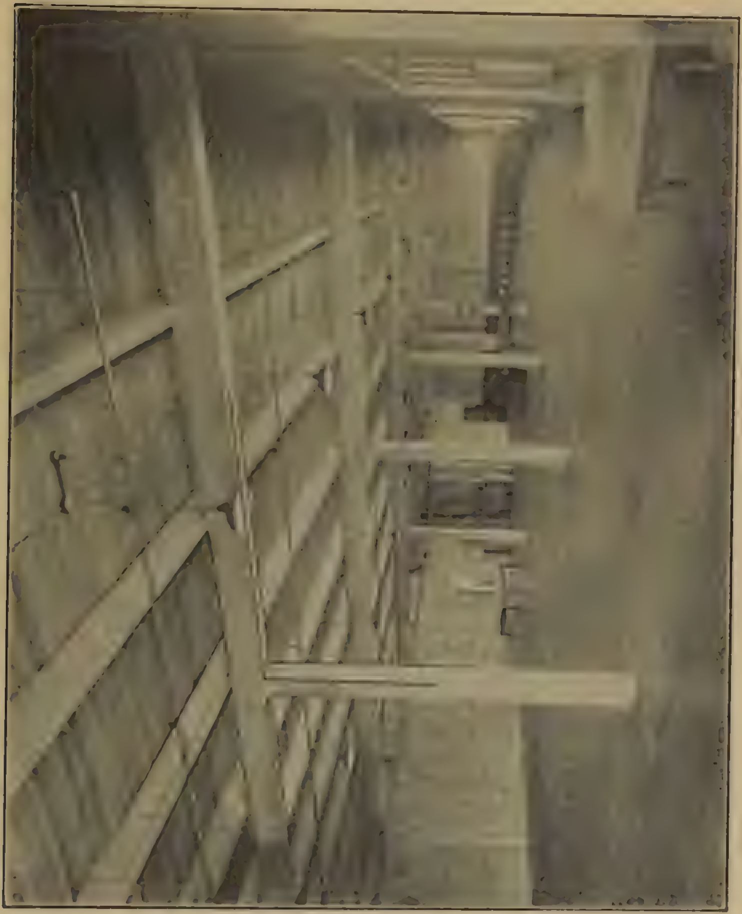


Fig. 75*b*.—Semi-mill Construction.

A type known as semi-mill construction consists of heavy plank laid flat upon large beams, spaced from 4 to 10 ft. apart on centers, and supported by girders. These girders are carried on wooden columns located as far apart as consistent with the general design of the building, distances from 20 to 25 ft. being not uncommon. Figure 75*b* illustrates this type. In this design the beams rest on the girders, which is much better than using hangers that are easily softened and give way if any material load is being carried.

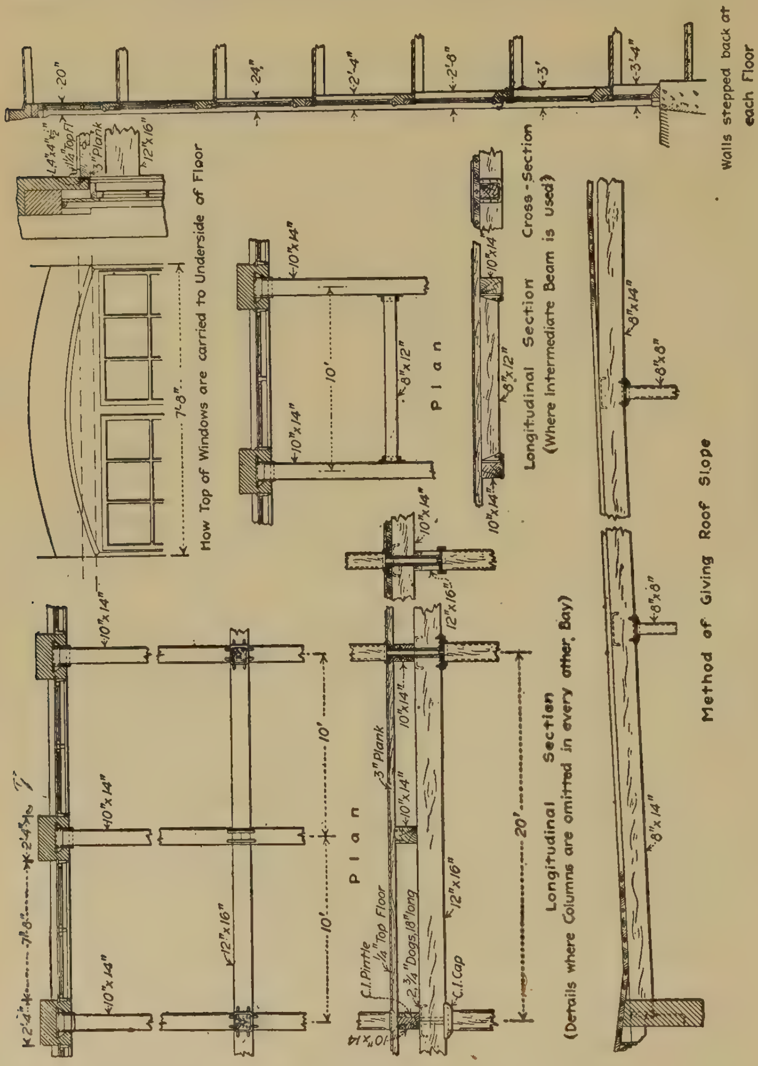

20"
24"
2'-4"
2'-8"
3'
3'-4"
Walls stepped back at each Floor
¼"x4"x4½"
1¼" Top Fl.
3" Plank
12"x16"
How Top of Windows are carried to Underside of Floor
7'-8"
10"x14"
10'
8"x12"
Plan
10"x14"
8"x12"
Longitudinal Section
Cross-Section
(Where Intermediate Beam is used)
8"x14"
8"x8"
Method of Giving Roof Slope
K2'-4"
7'-8"
2'-4"
10"x14"
12"x16"
10'
10'
Plan
3" Plank
C.I. Pintle
1¼" Top Floor
2¾" Dogs, 18" long
C.I. Cap
12"x16"
20'
Longitudinal Section
(Details where Columns are omitted in every other Bay)

FIG. 75*c*.—Details of Mill Construction.

Figures 75*c* and 75*d* illustrate some typical details in mill construction. It will be noted that the columns rest end to end on each other by the use of pintles. This eliminates the settlement—due to beam shrinkage—occurring when the columns are not continuous, due to setting the posts of one story on the beams which rest on the columns of the story below. Being of cast iron, the pintles are of small diameter and require only a small hole through the beams, half the hole being in one beam and half in the other. Hence the beams butt against each other and the ends of the beams are actually over the body of the column and are not supported by the overhanging ends of the column caps. Therefore, if a cap is

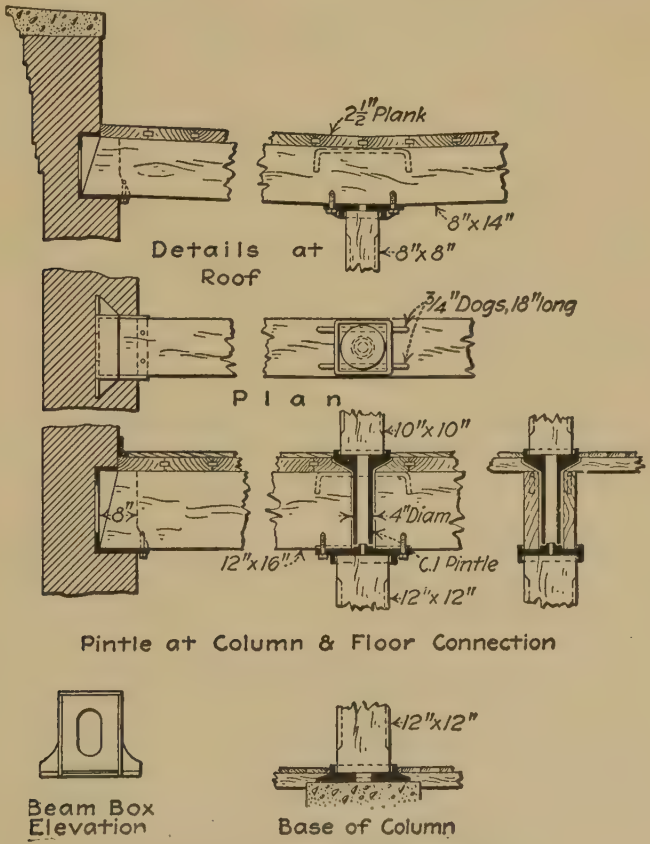

$2\frac{1}{2}$" Plank
8"x14"
8"x8"
Details at Roof
3/4" Dogs, 18" long
Plan
10"x10"
8"
4" Diam.
12"x16"
C.I Pintle
12"x12"
Pintle at Column & Floor Connection
12"x12"
Beam Box Elevation
Base of Column

FIG. 75*d*.—Details of Mill Construction.

burned off or breaks off the beams still remain in place. The use of pintles also permits the beams to act as struts to stiffen the building from one side to the other. The place where the pintle

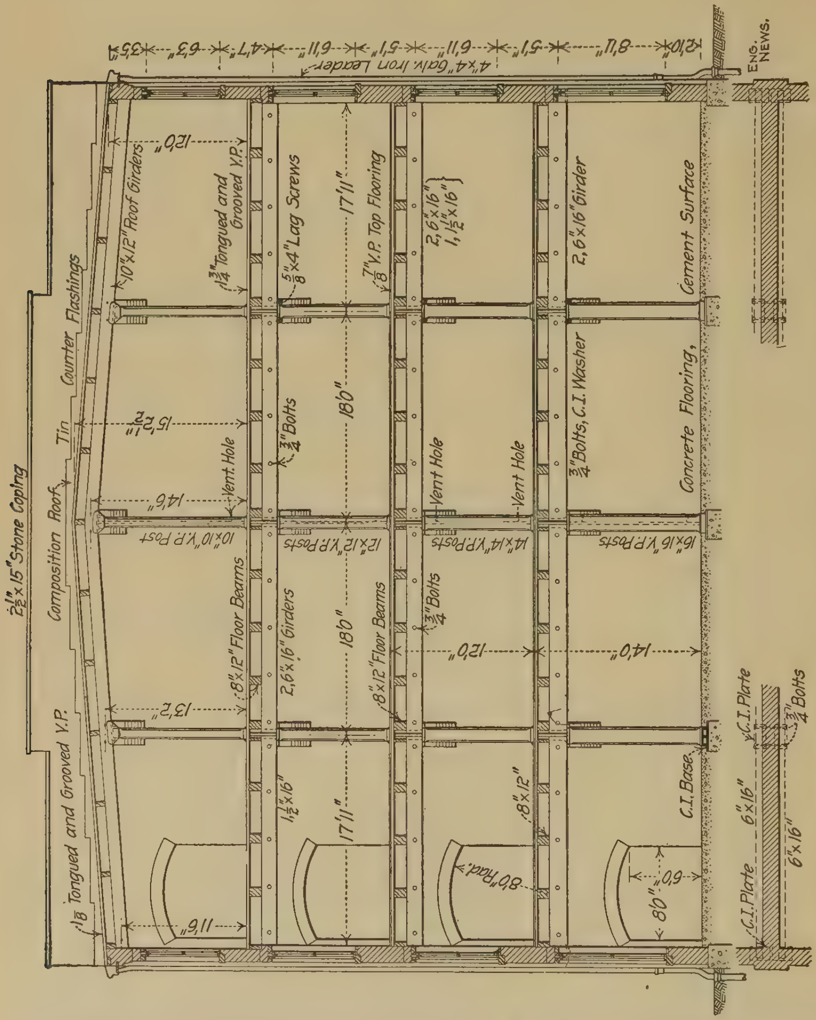

2½"x15" Stone Coping
Composition Roof
Tin
Counter Flashings
1⅞" Tongued and Grooved Y.P.
10"x12" Roof Girders
1¾" Tongued and Grooved Y.P.
⅝"x4" Lag Screws
⅞" Y.P. Top Flooring
2, 6"x16"
1, 1½"x16"
2, 6"x16" Girder
Cement Surface
Concrete Flooring,
¾" Bolts, C.I. Washer
Vent Hole
Vent Hole
Vent Hole
¾" Bolts
¾" Bolts
10"x10" Y.P. Post
12"x12" Y.P. Posts
14"x14" Y.P. Posts
16"x16" Y.P. Posts
8"x12" Floor Beams
8"x12" Floor Beams
2, 6"x16" Girders
1½"x16"
8"x12"
80" Rad.
C.I. Base
C.I. Plate
C.I. Plate
6"x16"
6"x16"
¾" Bolts
4"x4" Galv. Iron Leader
12'0"
15'2"
14'6"
13'2"
11'6"
17'11"
18'0"
18'0"
17'11"
12'0"
14'0"
6'0"
8'0"
ENG. NEWS.

FIG. 75e.—Transverse Section of Pattern Building, New Worthington Hydraulic Works.

enlarges at the top to take the upper column is the only part directly exposed to fire and here only at the rim.

Figures 75*e* and *f* show some details of modern slow-burning construction, as used in the pattern building of the New Worthington Hydraulic Works at Harrison, N. J. The building, which is 552 ft. long, is separated into four divisions by brick partition walls. The section shown is taken close to the middle wall. The first division is occupied by the office and drafting room, the second is the pattern shop proper, and the other two are used as pattern houses. The longitudinal spacings of the columns in these divisions are 20, 15, and 10 ft.

Where the post takes the rafter, use is made of the special caps shown in Figs. 75*g* and *h*, and at the bottoms of the columns are used base plates constructed as shown by Fig. 75*i*. Details of the wall connections for the girders and rafters are shown by Fig. 75*k*.

Over the office portion of the building the peaked roof shown by Fig. 75*e* is replaced by a saw-tooth roof to give light to the drafting room on the top floor. There are five bays of this roof, each bay running transversely across the building and consisting of two wall trusses and three intermediate trusses carrying purlins and roofing. The drawings of Fig. 75*f* show these trusses and their connections.

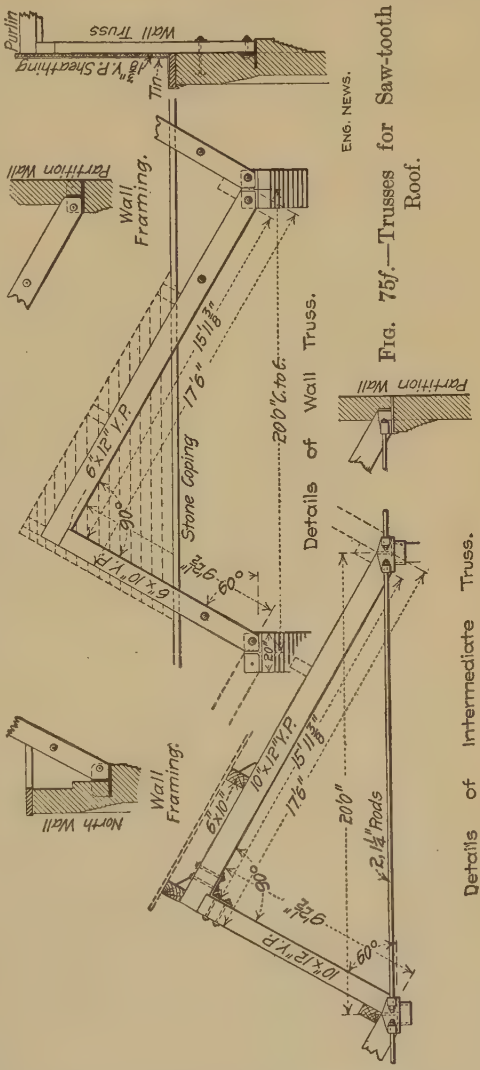

Purlin
Wall Truss
3/4" Y.P. Sheathing
Tin
Partition Wall
Wall Framing.
15'11 3/8"
17'6"
20'0" C. to C.
6"x12" Y.P.
6"x10" Y.P.
90°
60°
32'6"
Stone Coping
Details of Wall Truss.
ENG. NEWS.
Partition Wall
North Wall
Wall Framing.
6"x10"
10"x12" Y.P.
15'11 3/8"
17'6"
20'0"
2-1 1/4" Rods
90°
32'6"
10"x12" Y.P.
60°
Details of Intermediate Truss.

FIG. 75*f*.—Trusses for Saw-tooth Roof.

A number of special details used in slow-burning construction have been described in previous chapters, including pintles and post caps, beam hangers, beam anchors, etc., accompanied by notes on their relation to fire hazard. For additional information see the references given in Art. 82.

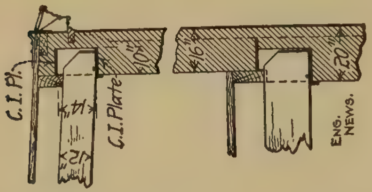


Fig. 75k. Beam Anchorage.

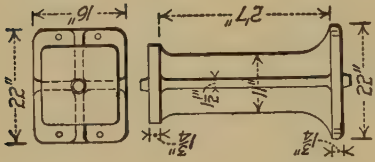


Fig. 75j. Post Cap.

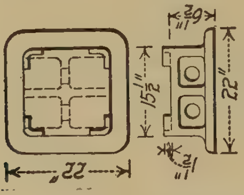
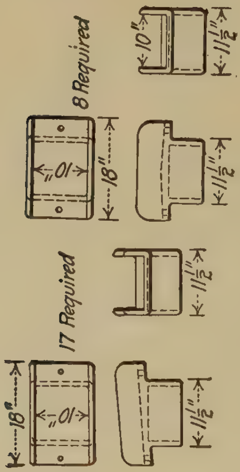


Fig. 75i. Base Plate.



Figs. 75g and h. Post Caps under Rafters.

Art. 76. Dry Rot in Mill Buildings

In recent years there has been a noticeable increase in the prevalence of dry rot in mill buildings, which threatens the economy and integrity of this type of construction. Wood infected by dry rot not only loses its strength but also ignites more easily than sound wood.

Rot in timber is due to a fungus growth which thrives best under a narrow range of moisture and temperature conditions. Wood constantly saturated with water will not rot, as proved by the piles under an old London bridge, which were found to be sound after being in use 600 years. In reconstruction work on the Campanile of St. Mark's in Venice in 1902 the old piles, placed 1002 years before, were in such a good state of preservation that they were used to support the reconstructed tower. Likewise, timber in dry air is immune from fungus attack. The roof beams of the Basilica at Rome were sound when 1000 years old. Figure 76*a* shows a $2\frac{1}{2}$-in. plank taken from the bottom of a tank, in which fungi decomposed only the narrow space in the middle, the inside, next to the water, being too wet, and the outside, next the air, being too dry.

Most dry rot is caused by two types of fungi, Merulius Lachrymans and Coniophora Cerebella, while damp rot is caused by members of the Polyporus family. Dry rot has been found in upper floors of reasonably dry mills, these fungi being able to thrive with a scant water supply. Damp-rot fungi are especially destructive in poorly ventilated basements. Dry-rot fungi grow by two methods of reproduction. The common reproductive spore grows over the surface of the plant, but under unfavorable conditions, such as insufficient moisture, the plant separates into small sections which can sprout and grow again when conditions are favorable, and can remain from two or four years in the resting state.

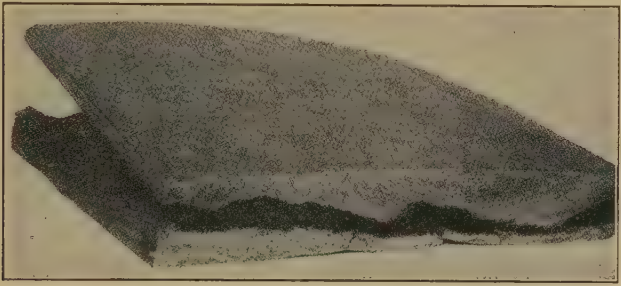


Fig. 76*a*.—Decay of Timber.

Rot disease is usually spread by direct contact, but living spores carried in the air can take root when they find a favorable resting place. However, when dry rot develops in a building it is usually due to the presence of the fungi on the timber at the time of construction. Usually planks and timber are run through a planer before delivery and this leaves a bright surface which can easily be mistaken for sound wood if it is used before drying has brought out the brown spots, which are recognized as rotten wood.

In studying the effect of moisture on dry rot one must carefully distinguish between relative and absolute humidity. Relative humidity is the ratio of complete saturation or the percentage of saturation of the air at the given temperature, but absolute humidity is the actual quantity of water in the air, regardless of the temperature. For example, if air at 70° Fahrenheit was found to contain four grains of water per cubic foot, the relative humidity would be 50 percent, since eight grains are required for saturation at this temperature.

Dry rot will grow on the surface of wood at atmospheric saturations of 96 to 100 percent and inside of large beams at much lower values. In one case a room was maintained at less than 70 percent relative humidity, yet a large proportion of the beams supporting the second floor were destroyed within two years.

Ventilation is generally the first preventive measure suggested. Dry wood placed in an atmosphere well below the moisture requirements of a given fungus is probably incapable of infection by that fungus; but ventilation does not necessarily cause drying, for drying depends on the relative humidity of the ventilating air and its temperature. A variation of temperature has a very marked effect. For example, with 70 percent saturation and a temperature of 80° Fahrenheit, a decrease of 12° at night would increase the relative humidity to 100 percent.

Often an infected building can be sterilized by use of its own heating system. Dry-rot fungus is very sensitive to heat, a temperature of 108° Fahrenheit for three hours, or 115° for one hour, being sufficient to kill it.

A heavy coat of paint may either accelerate or retard decay, depending on whether the wood is damp or dry at the time the painting is done.

Sometimes holes are bored in columns and beams for the object of preventing dry rot, but the common custom of boring green columns just before they are put in place in a building and using moist lumber for double beams leaves ideal places for the growth of fungi.

In new construction, timber having a maximum natural resistance to fungi should be used and this timber should be free from living fungi when placed in the structure. It is usually well worth the cost to heat the building as soon as possible after construction. This will hasten the drying of the timber and kill surface growths. Where manufacturing conditions demand a moist atmosphere some artificial antiseptic treatment should be used.

Southern pine is the most widely used timber for mill construction and it has been found that the denser woods of this species, well filled with rosin, are the most resistant to dry rot. The following specification has been proposed:

No part of the material shall have a density of less than 30 lb. per cu. ft. when tested by boring smooth holes 1 in. in diameter and 2 in. deep in the ends of the stick, drying to constant weight at 212° Fahrenheit, weighing the borings and computing the density from the volume of the hole. None of the heartwood shall show less than 4 percent rosin by weight, extracted with benzole from a sample taken as for the density determination. Heartwood shall show on all four faces of every stick and sapwood shall not extend more than 2 in. from the corner at any place. No timber shall contain knots greater than 1 in. in diameter, nor injurious shakes.

Antiseptic treatment is the only practicable procedure for lumber of doubtful durability. Coal-tar compounds are more used for preservative treatment than all other materials together. They act mechanically and chemically, serving as waterproofing as well as antiseptics. However, the disagreeable odor, greasy surface and increased inflammability resulting from their use make them objectionable, except possibly for basement floors and rooms.

Kyanizing by use of corrosive sublimate gives good results. The method of treatment employed is to soak the timber in a cold 1 percent solution in a concrete or wooden tank, the time of soaking in days equaling the thickness in inches plus 1.

For valuable material on the subject of this article the reader is referred to a pamphlet entitled " Dry Rot in Timber," prepared by F. J. Hoxie for the Associated Factory Mutual Fire Insurance Companies. Much of the material given above was taken from this pamphlet.

ART. 77. TRESTLE CONSTRUCTION

Various methods of framing the bents of wooden trestles are given in connection with the treatment of bolts and nuts in Art. 1; of drift bolts in Art. 9; of dowels in Art. 15; of metal straps and plates in Art. 19; of mortise-and-tenon joints in Art. 34; of wooden beams in Art. 41; of wooden posts in Art. 53; and of bolsters in Art. 54. The arrangement of the stringers is described in Art. 42 on packed stringers.

The sizes of timbers in trestle bents are determined chiefly by practical considerations apart from those required for the strength of the structure. For instance, the posts usually have a far greater degree of security than the caps, but since timbers have to be kept in stock and shipped frequently on telegraphic orders to replace an old structure which was suddenly destroyed by fire or flood, it is convenient to frame different members from sticks of the same size.

Standard railroad trestle designs, adopted in 1923 by the American Railway Engineering Association,[1] are shown in Plates VI, VII, VIII, and IX. The first illustration gives details for open-deck pile trestles and the second for open-deck frame trestles. Plate VIII shows the construction of multiple-story open-deck trestles. Ballasted-deck frame trestles are shown in Plate IX. All designs are based on Cooper's E 60 loading. Standards for E 45 loading, together with the above designs, may be found in the Proceedings of the above mentioned association, vol. 24, page 774, 1923. The E 45 designs differ from those based on E 60 chiefly in the use of one less pile or post and one less stringer under each rail.

Designs are given for panel lengths of 12, 14 and 16 ft. The 12-ft. panel lengths are desirable in level country, where bents are low; and the 16-ft. panels are most suitable in hilly and mountainous country requiring high bents.

The basic unit stresses used are given on the illustrations and agree closely with the recommended stresses of Art. 89 for dense-select Douglas fir and southern pine, except for horizontal shear. In preparing designs the weight of timber and hardware was taken as 4.5 lb. per board foot. The weight of rails and fastenings was taken as 150 lb. per linear foot of deck.

Where concrete footings are used the posts bear directly on the concrete, thereby eliminating the sill. Where a sill is used it usually shows weakness by decay or compression. Horizontal braces above the concrete tie the posts together and dowels are used to hold the bents in line on the concrete.

The ballast-deck trestle is more fire resistant than the open-deck trestle, requires less maintenance, has a better and safer appearance, rides better, and has a longer life. Open-deck structures are sometimes made partially fire resistant from hot cinders by (1) covering decks with sheet metal, (2) covering caps and stringers with sheet metal, and (3) using a solid floor covered with gravel. The second method is illustrated in Fig. 77*a*, where galvanized sheet iron is laid on both stringers and cap and bent down over the edges. The sheets should be about 11 in. wider than the timbers covered, and of about 22 to 27 gage in thickness, having a lap longitudinally of about 6 in. They should be protected from rust by heating the iron and painting it with tar. The iron dowels connecting the ties to the stringers will hold the sheet iron in place. The sheet iron also performs another important function in protecting the tops of stringers and caps from water and thereby materially prolonging the life of these timbers. Since the strength of stringers is impaired to some extent by preservative processes the importance of protecting them from decay by cover deserves increasing consideration.

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[1] Proceedings A. R. E. A., 1923.

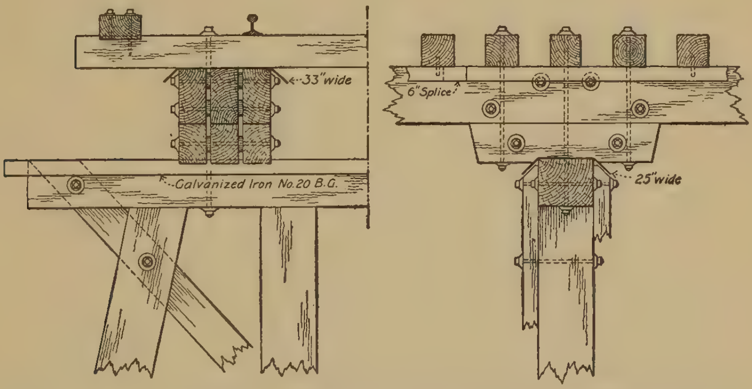
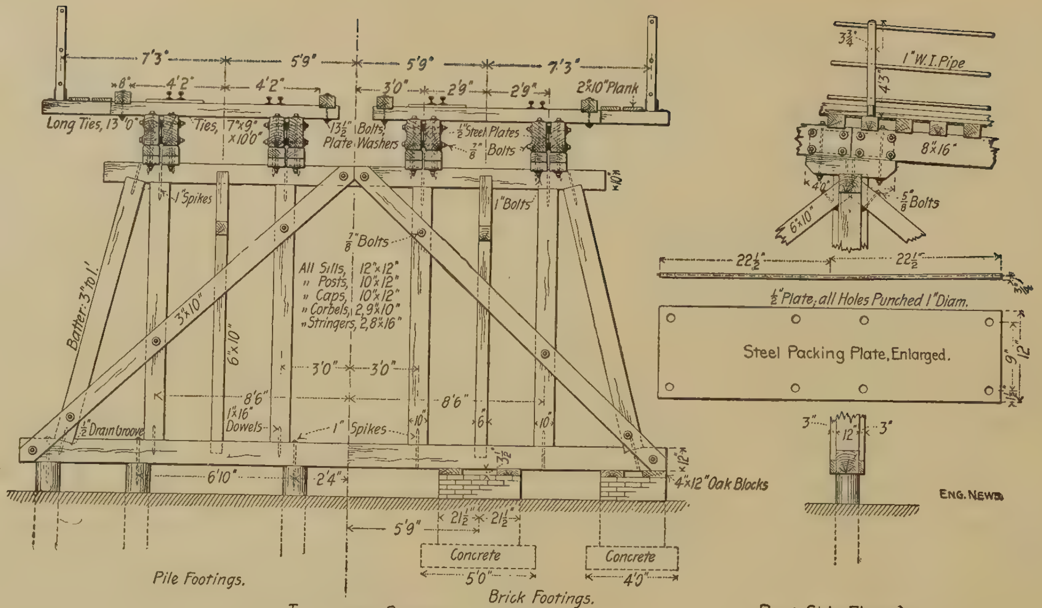


FIG. 77*a*.—Iron Covering on Stringers and Cap to Prevent Fire and Decay.

Figure 77*c* illustrates some bridge trestle construction as used by the Louisiana Highway Commission. This design provides for a live load consisting of two 10-ton motor trucks plus 30 percent for impact. The five-pile bents are spaced 19 ft. center to center, all piles being vertical. In addition to the drift bolts, each cap is fastened to the two outer piles by pairs of hook-end bars fitted over horizontal screw bolts through the pile and cap. Between each pile head and cap is placed a sheet of No. 28 gage galvanized iron, which is bent down to shed water. Two types of wearing surface are used: gravel and rock asphalt. All structural


7'3"
5'9"
5'9"
7'3"
8"
4'2"
4'2"
3'0"
2'9"
2'9"
2"x10" Plank
Long Ties, 13'0"
Ties, 7"x9" x10'0"
13½" Bolts, Plate Washers
½" Steel Plates
⅞" Bolts
10"
1" Spikes
1" Bolts
⅞" Bolts
Batter: 3" to 1'
3"x10"
6"x10"
All Sills, 12"x12"
" Posts, 10"x12"
" Caps, 10"x12"
" Corbels, 2, 9"x10"
" Stringers, 2, 8"x16"
3'0"
3'0"
8'6"
8'6"
1"x16" Dowels
1" Spikes
½" Drain Groove
10"
6"
10"
3½"
12"
6'10"
2'4"
4"x12" Oak Blocks
5'9"
21½"
21½"
Concrete
5'0"
Concrete
4'0"
Pile Footings.
Brick Footings.
3¾"
1" W.I. Pipe
4'3"
8"x16"
4"
6"x10"
⅝" Bolts
22½"
22½"
½" Plate, all Holes Punched 1" Diam.
Steel Packing Plate, Enlarged.
9"
12"
3"
12"
3"
ENG. NEWS.

Transverse Section.

Part Side Elevation.

Fig. 77b.—Details of Bent Used in Trestle at Long Beach, N. J., for the Atlantic Highlands, Red Bank, and Long Branch Electric Railway.

Noticeable features are the use of square ends for posts held in place by dowels and the entire absence of mortise-and-tenon joints. The ends of the posts and the triangular notches cut for them were thoroughly coated with asphaltum. At alternate bents, steel packing plates are used instead of wooden ones and are separated from the stringers by washers, and at the other bents, plate spacers 12 inches long are connected in a similar manner. See *Eng. News*, Vol. 39, page 21, Jan. 13, 1898.

material is southern pine, the piles and timbers, with the exception of the railings, being creosoted by the empty-cell process, giving 8 lb. of oil per cubic foot for the floor planks and 12 lb. for all other material. The railings are painted white.

Figure 77*d* shows the braced towers of the stairway approach to the Rhawn Street timber viaduct in Philadelphia, over Pennypack Creek. The utmost economy in the design was exercised to diminish the first cost, and at the same time to meet all the requirements of traffic. The wood is yellow pine and the bracing is mainly bolted in place. The lower timbers in the side diagonals and the diagonals of the transverse bracing are connected to the post by single step joints. Figure 77*e* also shows the splices in the bottom longitudinal braces in which tabled fish plates are employed.

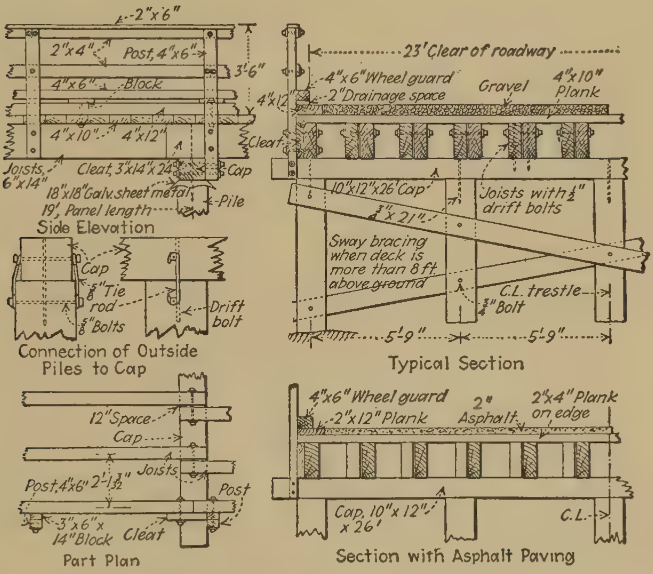

2"x6"
2"x4"
Post, 4"x6"
3'-6"
4"x6"
Block
4"x12"
4"x10"
4"x12"
Cleat
Joists, 6"x14"
Cleat, 3"x14"x24"
Cap
18"x18" Galv. sheet metal
Pile
19', Panel length
Side Elevation
Cap
5/8" Tie rod
5/8" Bolts
Drift bolt
Connection of Outside Piles to Cap
12" Space
Cap
Post, 4"x6"
2'-1 1/2"
Joists
Post
3"x6"x14" Block
Cleat
Part Plan
23' Clear of roadway
4"x6" Wheel guard
2" Drainage space
Gravel
4"x10" Plank
10"x12"x26' Cap
3"x21"
Joists with 1/2" drift bolts
Sway bracing when deck is more than 8 ft. above ground
C.L. trestle
3/4" Bolt
5'-9"
5'-9"
Typical Section
4"x6" Wheel guard
2"x12" Plank
2" Asphalt
2"x4" Plank on edge
Cap, 10"x12"x26'
C.L.
Section with Asphalt Paving

FIG. 77*c*.—Trestle with Paved Floor.

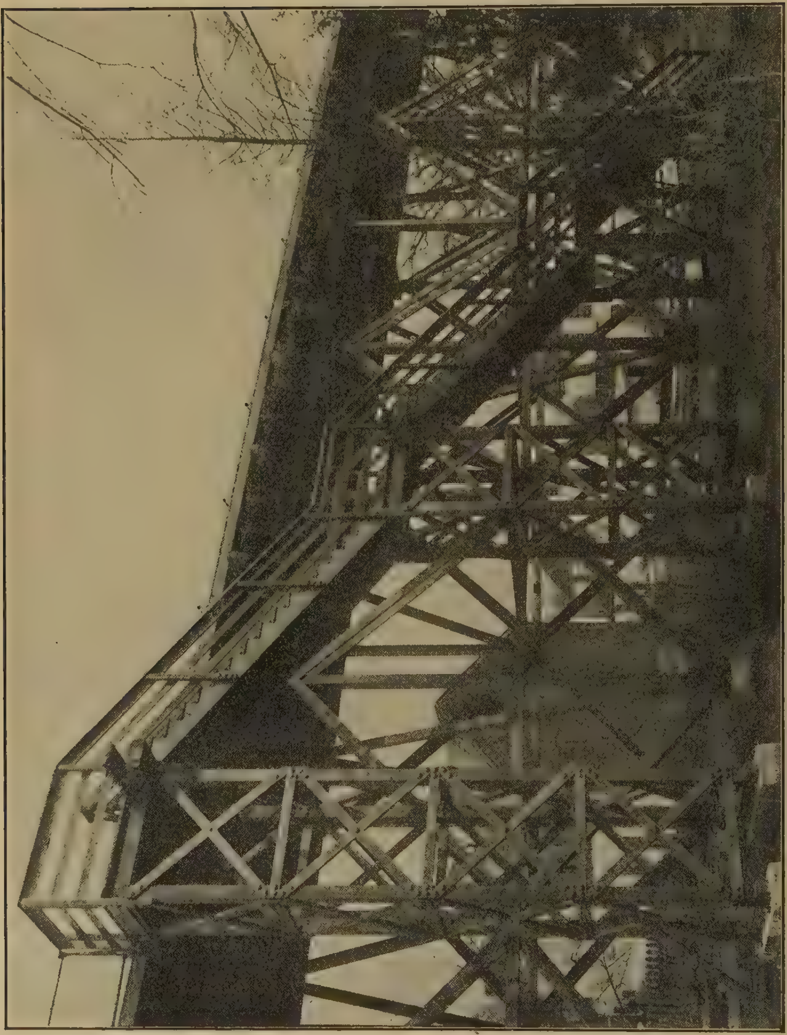


Fig. 77*d*.—Stairway Approach to Rhawn Street Viaduct, Philadelphia, 1900.

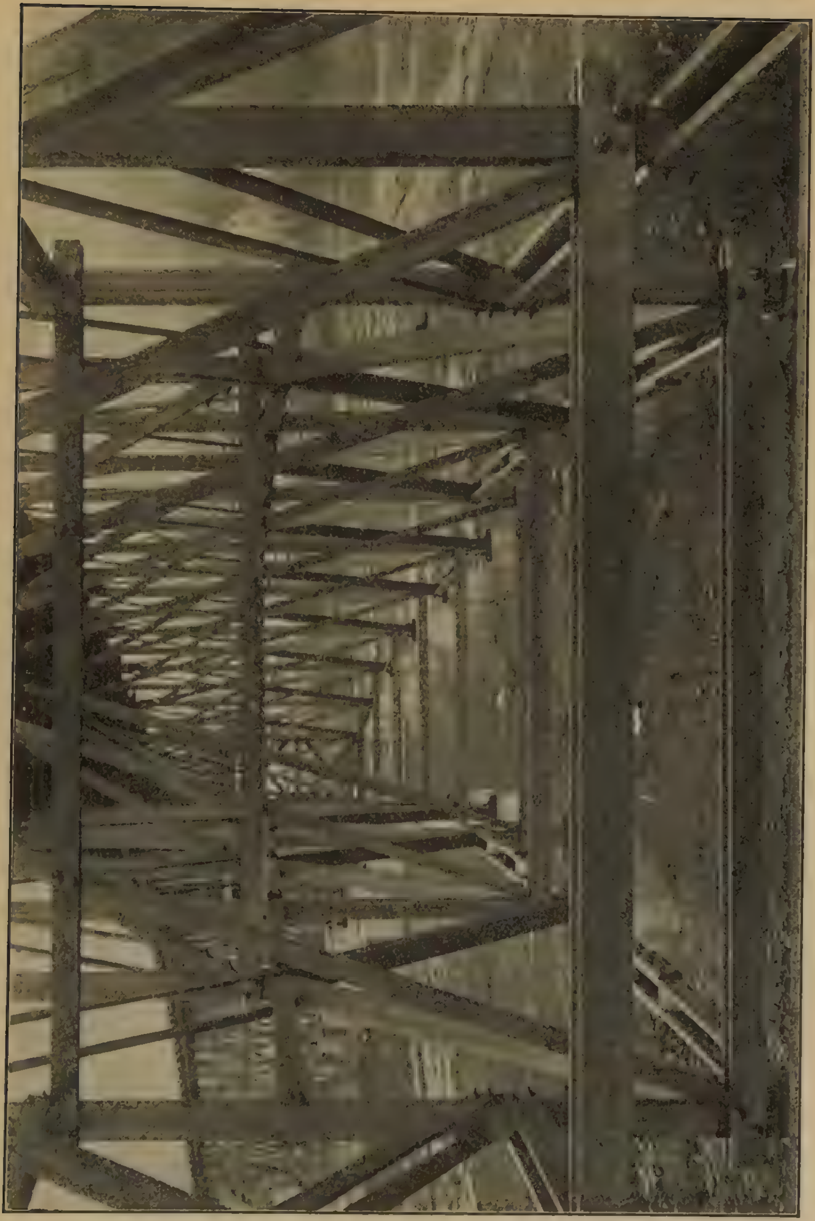


FIG. 77c.—Highway Viaduct on Rhawn Street, Philadelphia, Pa., 1900.

ART. 78. TRUSS BRIDGES

Figure 78*a* shows the standard design of a 60-ft. span pony Howe truss bridge of the Utah state highway department, using a composite lower chord. This bridge is designed for a dead load of 680 lb. per lin. ft., a uniform live load of 100 lb. per sq. ft., and a concentrated live load of an 18-ton roller. The impact allowance is 25 percent of the live load for the floor system and 18 percent for the trusses.

In the lower chord all the tension is carried by the steel plate laid on top of the two 4- by 6-in. timbers, the latter giving the necessary stiffness. This bridge contains 14,400 board feet of timber, 4250 lb. of metal in angles, plates, bolts and nails, 1850 lb. of iron castings, and 730 lb. of hangers and nuts.

In Fig. 78*b* is shown the standard 170-ft. span through Howe truss bridge of the Washington state highway department. It will be noted that the lower lateral system of bracing is composed of rods.

A typical railroad bridge design as used on the Alaskan Government Railroad is shown in Fig. 78*c*, this design being based on Cooper's E 50 loading, no allowance being made for impact. The top and bottom chords are composed of four sticks 8 by 14 in. and 8 by 16 in., respectively, with cast-iron angle blocks for the truss members and transverse steel channels as bearings for the washers of the vertical tension roods. Floor-beams are composed of pairs of 12- by 24-in. timbers, supporting track stringers composed of pairs of 8- by 12-in. timbers.

Figure 78*d* gives a side view of a short-span single-track railroad bridge in the Rock Mountain region of British Columbia. Cast-iron angle blocks were used at the intermediate joints of the upper and lower chords, while the single step joint like Fig. 35*a* was used at both ends of the inclined end posts. The sticks in both chords are continuous. A bearing block for the diagonal strut was inserted at the end joint of each upper chord. As the cross-ties rest on the lower chord, the chord members are subject to combined tension and flexure.

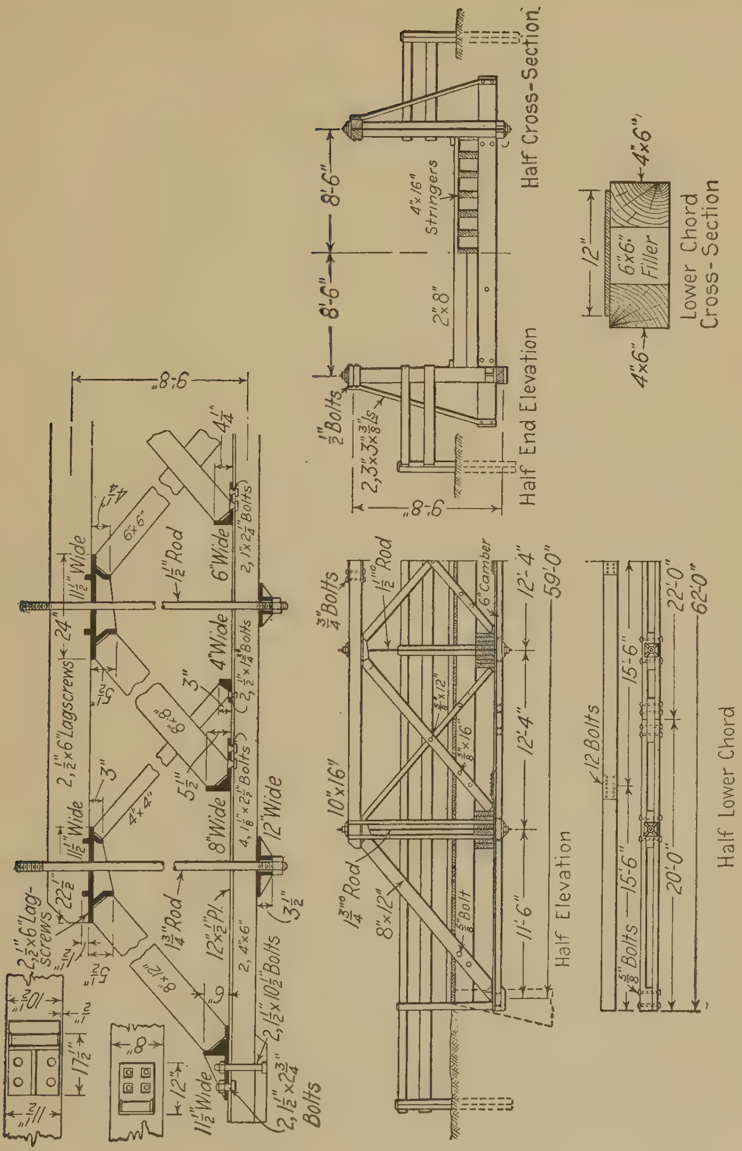

Half Cross-Section
Half End Elevation
8'-6"
8'-6"
4"x16" Stringers
2"x8"
1/2" Bolts
2, 3"x3"x3/8" Ls
8'-6"
Lower Chord Cross-Section
12"
6"x6" Filler
4"x6"
4"x6"
3/4" Bolts
1 1/2" Rod
6" Camber
12'-4"
59'-0"
12'-4"
10"x16"
5/8"x12"
5/8"x16"
1 3/4" Rod
8"x12"
5/8" Bolt
11'-6"
Half Elevation
12 Bolts
15'-6"
15'-6"
5/8" Bolts
22'-0"
62'-0"
20'-0"
Half Lower Chord
8'-6"
4 1/4"
4 1/4"
6"x6"
1 1/2" Rod
1 1/2" Wide
6" Wide
2, 1"x2 1/4" Bolts
24"
2, 1/2"x6" Lagscrews
5 1/2"
4" Wide
3"
8"x8"
2, 1/2"x1 3/4" Bolts
3"
5 1/2"
4"x4"
1 1/2" Wide
8" Wide
4, 1/8"x2 1/2" Bolts
12" Wide
22 1/2"
2, 1/2"x6" Lag-screws
1 3/4" Rod
12"x1/2" Pl.
2, 4"x6"
3 1/2"
5 1/2"
1 1/2"
10 1/2"
8"x12"
6"
2, 1 1/2"x10 1/2" Bolts
17 1/2"
8"
12"
11 1/2" Wide
2, 1 1/2"x2 3/4" Bolts
11 1/2"

FIG. 78a.—Pony Howe-truss Highway Bridge.

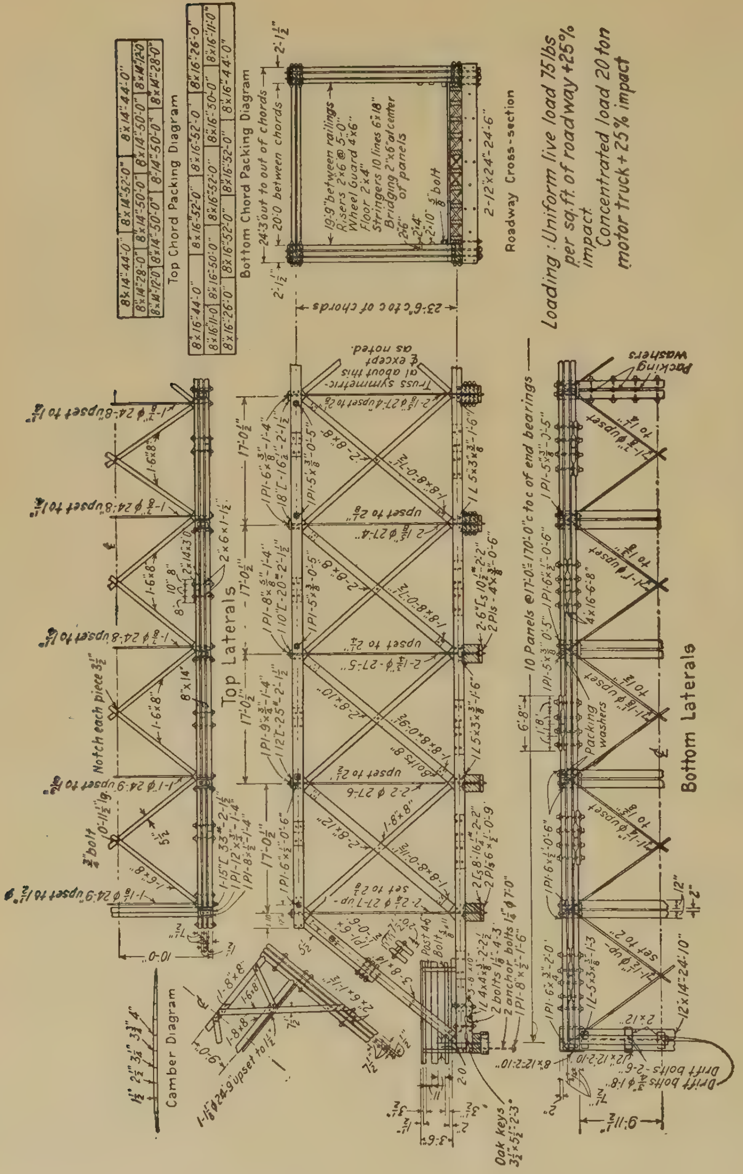

Top Chord Packing Diagram
8"x14"-44'-0" | 8"x14"-52'-0" | 8"x14"-44'-0"
8"x14"-28'-0" | 8"x14"-50'-0" | 8"x14"-50'-0" | 8"x14"-12'-0"
8"x14"-12'-0" | 8"x14"-50'-0" | 8"x14"-50'-0" | 8"x14"-28'-0"
Bottom Chord Packing Diagram
8"x16"-44'-0" | 8"x16"-52'-0" | 8"x16"-52'-0" | 8"x16"-26'-0"
8"x16"-11'-0" | 8"x16"-50'-0" | 8"x16"-52'-0" | 8"x16"-50'-0" | 8"x16"-11'-0"
8"x16"-26'-0" | 8"x16"-52'-0" | 8"x16"-52'-0" | 8"x16"-44'-0"
24'-3" out to out of chords
20'-0" between chords
2'-1½"
19'-9" between railings
Risers 2"x6" @ 5'-0"
Wheel Guard 4"x6"
Floor 2"x4"
Stringers 10 lines 6"x18"
Bridging 2"x6" at center of panels
⅝" bolt
2-12"x24"-24'-6"
Roadway Cross-section
23'-6" c to c of chords
Loading: Uniform live load 75 lbs per sq. ft. of roadway + 25% impact.
Concentrated load 20 ton motor truck + 25% impact
Truss symmetrical about this ℄ except as noted.
Top Laterals
Notch each piece 3½"
10 Panels @ 17'-0" = 170'-0" c to c of end bearings
Packing washers
Bottom Laterals
Camber Diagram
Post 4½
Oak keys 3½"x5½"-2'-3"
Drift bolts ¾" ϕ 1'-8"
Drift bolts -2'-6"
12"x14"-24'-10"
2 anchor bolts 1⅛"-4'-3"
17'-0½"
6'-8"
9'-11¾"
3'-6"

FIG. 78b.—Howe-truss Highway Bridge.

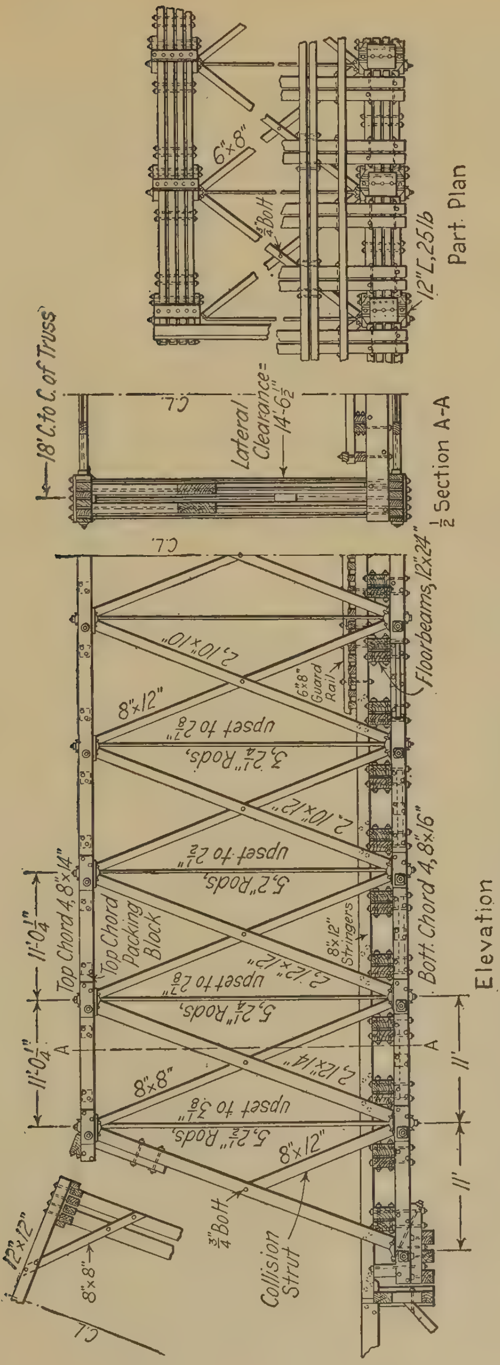

18' C. to C. of Truss
C.L.
Lateral Clearance = 14'-6½"
6"×8"
¾" Bolt
12" [, 25 lb
Part Plan
½ Section A-A
Floorbeams, 12"×24"
2, 10"×10"
8"×12"
6"×8" Guard Rail
upset to 2⅞"
3, 2¼" Rods,
2, 10"×12"
upset to 2½"
5, 2" Rods,
Top Chord 4, 8"×14"
11'-0¼"
Top Chord Packing Block
8"×12" Stringers
Bott. Chord 4, 8"×16"
2, 12"×12"
upset to 2⅞"
5, 2¼" Rods,
A
A
11'-0¼"
8"×8"
2, 12"×14"
upset to 3⅛"
5, 2½" Rods,
8"×12"
11'
11'
¾" Bolt
Collision Strut
12"×12"
8"×8"
C.L.
Elevation

FIG. 78c.—Howe-truss Railroad Bridge.

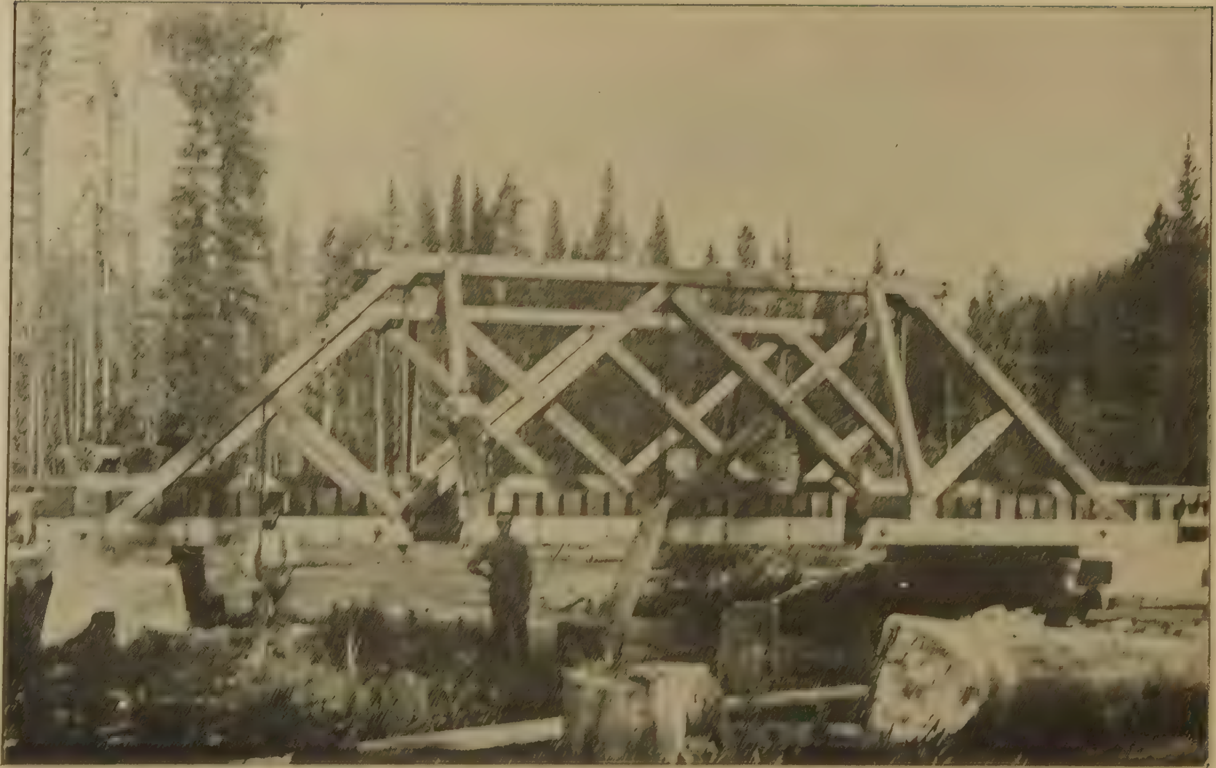


Fig. 761.—Pony Howe Truss Bridge at Crow's Nest Line, Canadian Pacific Railway, Span 50 feet.

ART. 79. MISCELLANEOUS BRIDGES

The Municipal Ferry transfer bridges shown in Fig. 79*a* are built entirely of wood except for the bolts and a few tie rods used for connections. They are made unusually heavy and have four arch ribs or quasi-trusses of the bowstring form to support the platform. The lower chords are heavy continuous timbers, and on account of their flexural strength no diagonals are inserted in the trusses. The wooden floor is supported on two tiers of heavy longitudinal and transverse timbers spaced close together, bolted at every intersection and suspended at seven panel points from each arched chord by a pair of screw rods.

The river end of the floor is curved exactly to fit the end of the ferry boat, and is faced with vertical 4-in. oak lagging and ½-in. iron bands spiked to horizontal oak saddle pieces. These have almost continuous backing, with longitudinal and inclined struts bolted to the main girders in the floor, so as to distribute thoroughly any impact received. The curved chords are built up of 3-in. planks with lapped joints, which are bolted together with ¾-in. bolts spaced about 12 in. apart. Their ends are stepped and bolted to the 8- by 16-in. bottom chord timbers. The 16- by 16-in. transverse oak timber at the shore end of the platform is thoroughly braced and connected to the longitudinal timbers and upper chords, and has a cylindrical lower surface faced with a steel plate, which revolves in a corresponding steel-faced quoin fixed to the tops of the platform piles.

The outer end of the bridge is supported on a pontoon, and is automatically adjusted by the tide itself, except for slight differences in elevation, which are controlled by windlasses. The pontoons are built entirely of wood and extend nearly across the full width of the bridge, which takes bearing on them with each of its six principal longitudinal beams, and are thoroughly bolted through its principal deck members. A section of the pontoon is given in Fig. 79*b*. The transverse vertical frames, 27 in. apart, are made with 6- by 8-in. lap-jointed yellow pine timbers, with hackmatack knees and 1¼-in. treenails at their intersections. They are covered with an inner longitudinal planking of 3-in. yellow pine and an outer transverse planking of 2-in. spruce, both caulked and separated by a layer of ship felt. The top is covered with a tight deck of 4-in. planks. The framing of the piers, ferry

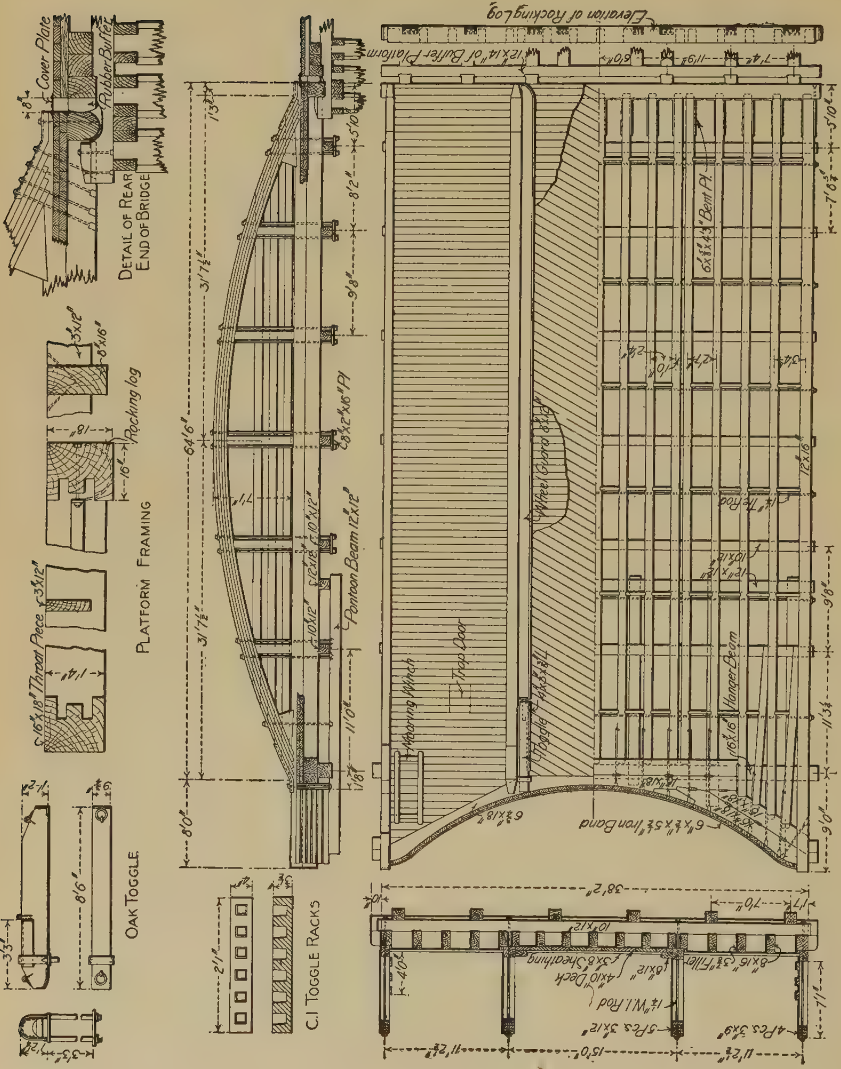

Cover Plate
Rubber Buffer
Detail of Rear End of Bridge
Platform Framing
Rocking log
Throat Piece
Oak Toggle
C.I Toggle Racks
Elevation of Rocking Log
12"x14" of Buffer Platform
Pontoon Beam 12"x12"
64'6"
31'7½"
Bent Pl.
Hanger Beam
Wheel Guard
Toggle
Trap Door
Mooring Winch
Tie Rod
Iron Band
38'2"
10"x12"
5x8 Sheathing
4"x10" Deck
1½" W.I. Rod
5 Pcs. 3"x12"
4 Pcs. 3"x9"
15'0"
11'2½"

FIG. 79a.—Framing of Transfer Bridge at Municipal Ferry House near Battery Park, New York City. Built in 1907.

racks, etc., is described and illustrated in an article in Engineering News, vol. 58, page 525, November 7, 1908, from which the information on the ferry bridge and pontoon is taken.

The temporary arched bridges of short span over canals and waterways at the Pan-American Exposition were constructed on the site by carpenters and laborers using ordinary commercial sizes and moderate lengths of pine and hemlock timber, mostly 4 by 12 in. or less in cross-section, bolted together without special forgings or any but a very few of the simplest castings. The simple character of the details is indicated in Fig. 79*c*. The conditions required considerable clearance for pleasure boats, to keep the roadway grade as low as possible, and to be safe and solid under a heavy live load of pedestrians.

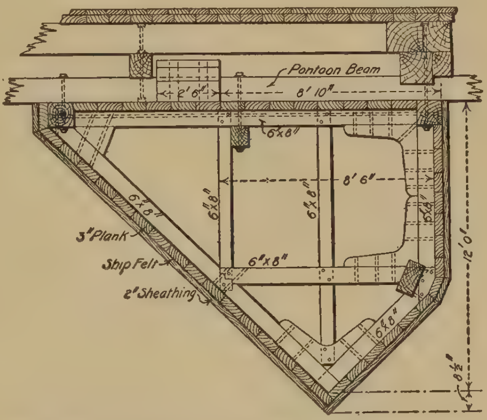


Fig. 79*b*.—Section of Transfer Bridge Pontoon.

The principal framework consisted of nine arched ribs, spaced 6 to 6½ ft. between centers, and having solid spandrel bracing extending to the longitudinal stringers, which were nearly parallel to the grade of the roadway. The ribs thus resembled pairs of solid triangular girders continuous over the crown, and having solid end bearings, with long rigid connections with the vertical abutment piles. The ribs were connected together by the floor joists and floor planks, and at the intrados by horizontal transverse struts and the sheathing which covered them. This construction shows that the ribs did not act as true arches, but as containing a combination of elements consisting of a beam with fixed ends, a cantilever, and an arch. Each rib was composed of four planks bolted together and keyed with 2-in. round white-oak keys. Each end of the rib took bearing on a hollow cast-iron skewback block which was notched into a longitudinal oak cap, and this in turn was mortised over one pile and connected to the other by a plaster joint. The manner in which the bridge was finished to harmonize with the architectural motives of the buildings and the landscape features of the vicinity are described in Engineering Record, vol. 44, page 394, October 26, 1901.

Plate X shows the details of the swing span of a temporary wooden bridge which was built in 1905 and was in service during the construction of the steel rolling lift bridge on 22d Street, Chicago, which was completed in 1906. This span was subsequently used at Ashland Avenue and then at 35th Street, while another one built on the same plans was in service at Archer Avenue. The first temporary swing bridge of this type was in service during the reconstruction of the Northwestern Avenue bridge, which was completed in 1904, others being used at North

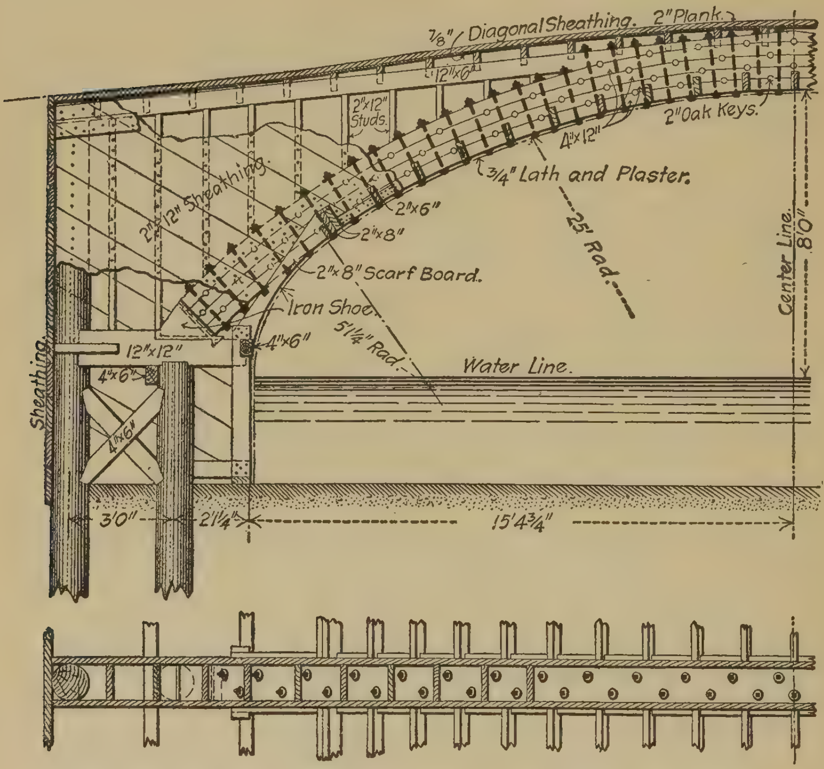
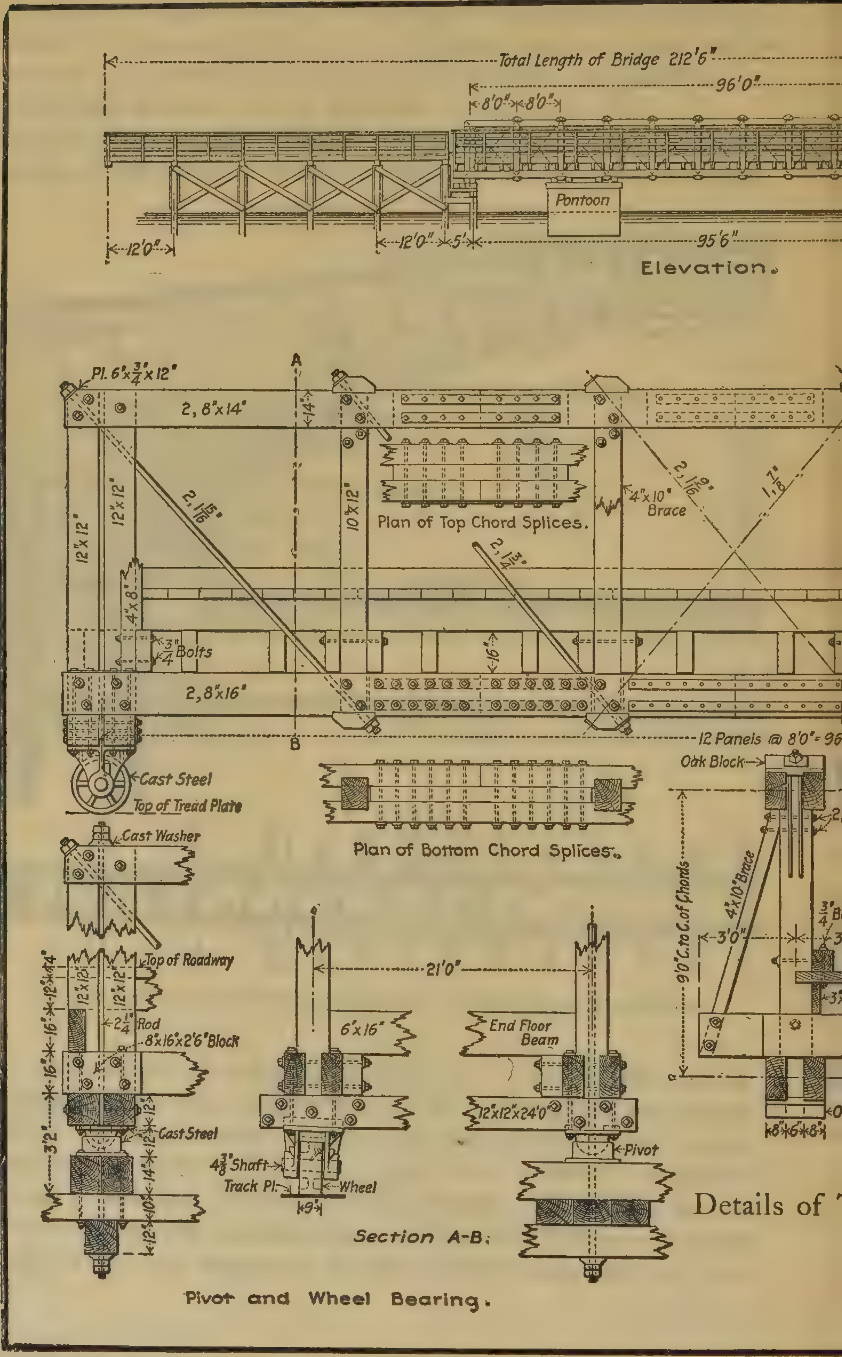

7/8" Diagonal Sheathing.
2" Plank.
12"x6"
2"x12" Studs.
2" Oak Keys.
4"x12"
3/4" Lath and Plaster.
2"x12" Sheathing.
2"x6"
2"x8"
2"x8" Scarf Board.
Iron Shoe.
25' Rad.
5 1/4' Rad.
4"x6"
12"x12"
4"x6"
4"x6"
Water Line.
Center Line.
8'0"
Sheathing.
3'0"
2'1 1/4"
15'4 3/4"

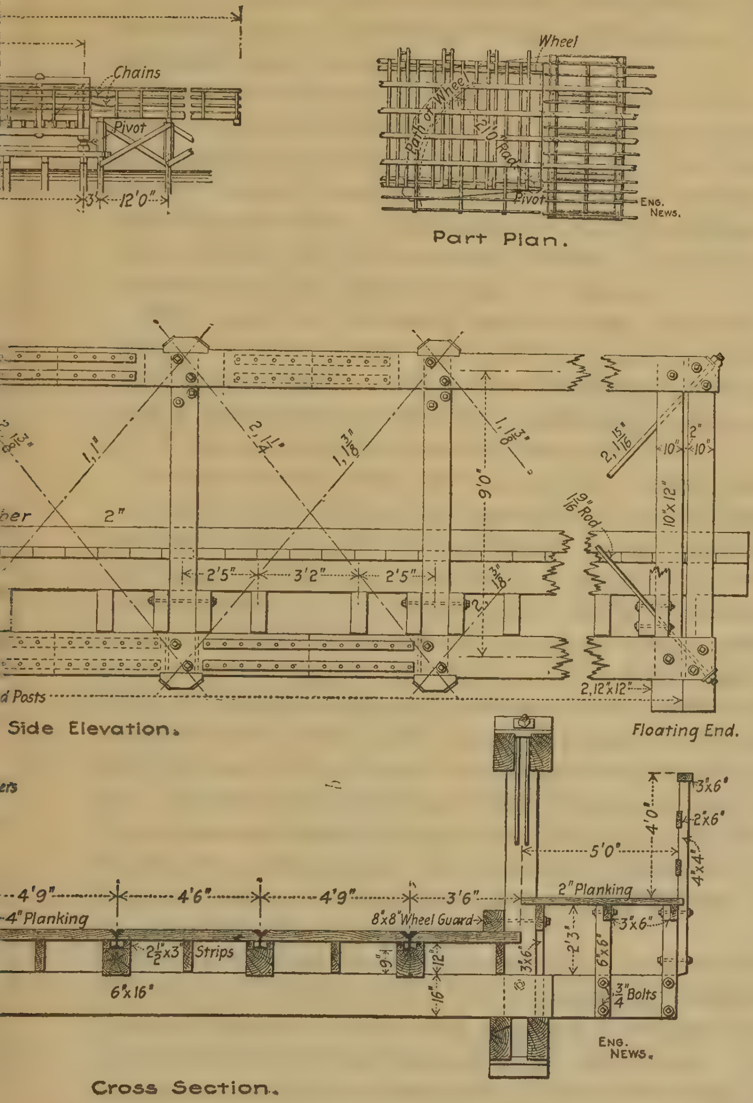
FIG. 79c.—Wooden Bridge Construction, Pan-American Exposition.


Total Length of Bridge 212'6"
96'0"
8'0"
8'0"
Pontoon
12'0"
12'0"
5'
95'6"
Elevation.
A
Pl. 6"x$\frac{3}{4}$"x12"
2, 8"x14"
14"
12"x12"
12"x12"
2, 1$\frac{15}{16}$"
10"x12"
Plan of Top Chord Splices.
2, 1$\frac{9}{16}$"
4"x10" Brace
1, 1$\frac{7}{8}$"
2, 1$\frac{3}{4}$"
4"x8"
$\frac{3}{4}$" Bolts
16"
2, 8"x16"
B
12 Panels @ 8'0" = 96
Cast Steel
Top of Tread Plate
Plan of Bottom Chord Splices.
Oak Block
Cast Washer
4"x10" Brace
9'0" C. to C. of Chords
3'0"
$\frac{3}{4}$" B
Top of Roadway
14"
12"
16"
16"
3'2"
12"x12"
12"x12"
2$\frac{1}{4}$" Rod
8"x16"x2'6" Block
12"
Cast Steel
12"
14"
10"
12"
21'0"
6"x16"
End Floor Beam
12"x12"x24'0"
4$\frac{3}{8}$" Shaft
Track Pl.
Wheel
9"
Pivot
Section A-B.
8"
6"
8"
Pivot and Wheel Bearing.

Details of

Facing Page 270.

Art. 79. Plate X.


Chains
Pivot
3'
12'0"
Wheel
Path of Wheel
1 1/4" Rod
Pivot
Eng. News.
Part Plan.
1 3/8"
1"
2, 1 1/4"
1, 1 3/8"
1, 1 3/8"
9'0"
2, 15/16"
2"
10"
10"
10"x12"
9/16" Rod
ber
2"
2'5"
3'2"
2'5"
3/8"
2, 12"x12"
d Posts
Side Elevation.
Floating End.
ers
3"x6"
2"x6"
4'0"
5'0"
4"x4"
4'9"
4'6"
4'9"
3'6"
2" Planking
4" Planking
8"x8" Wheel Guard
3"x6"
2 1/2"x3" Strips
9"
12"
3"x6"
2'3"
6"x6"
6"x16"
16"
3/4" Bolts
Eng. News.
Cross Section.

ary Bridge with Pontoon Swing Span over South Branch
Chicago River at Twenty-Second Street.

an 96 feet. Built in 1905.

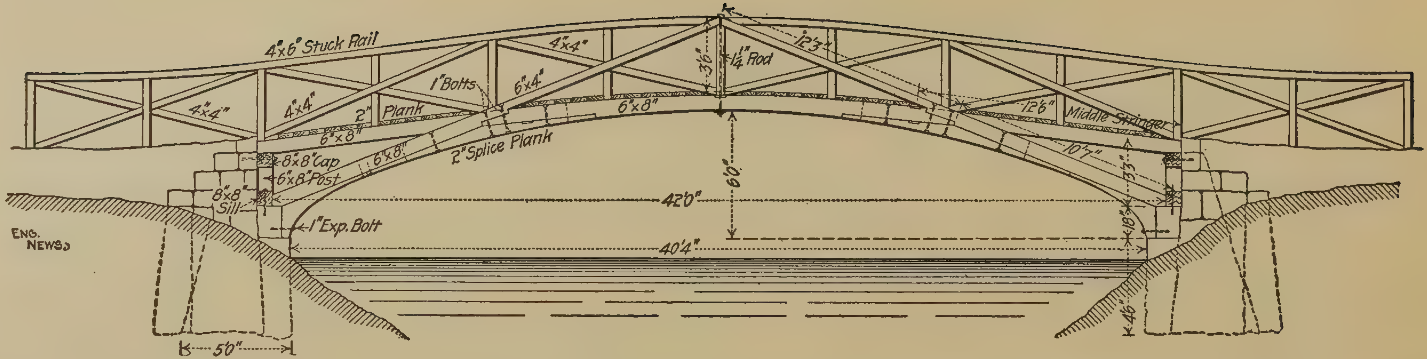
western Avenue, North Avenue, and Kinzie Street. The design was made by the Division of Bridges and Harbor of the Bureau of Engineering of the City of Chicago.

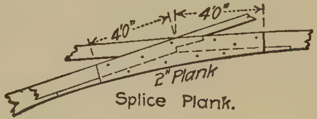
As indicated in the elevation and part plan, it has a pivot and wheel bearing, respectively, at the two corners of one end and is supported by a fixed bearing at the other end when closed, and by a scow or pontoon when swinging. It carried a double-track electric railway and was located a little distance upstream from the site of the permanent bridge. The clear waterway provided was 72 ft. when the span was opened.

The weight of the swing span is about 100 tons, and it is designed to carry a live load of 2000 lb. per lin. ft., with local wheel loads of 5000 lb. The trusses, floor beams, and pontoon are built of yellow pine, the caps, pivot platform, and roadway flooring are of white oak, and the piles of Norway pine.

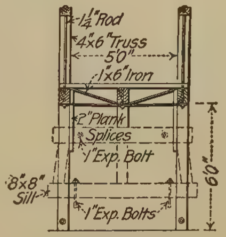
It is to be noted that the truss is of the Pratt type instead of the Howe type and that the connections for the diagonal rods are made very simple by providing bearing blocks outside of the chords. Each of the chords is composed of two lines of timber spaced a short distance apart, the rods passing between them. The ends of the verticals also pass between, whereas their shoulders bear against the narrow sides of the chord timbers. Each of the steel fish plates in the lower chord splices is $3\frac{1}{2}$ by $\frac{1}{2}$ in., the filler block between the two chord timbers is 6 by 16 in., and the bolts are 1 in. in diameter inserted in their holes with a driving fit. The fish plates on the upper chord splices are only $\frac{3}{8}$ in. thick. The remaining details of framing are indicated on the drawing and require no explanation. For additional information regarding the cost, method of operation, etc., see Engineering News, vol. 54, page 698, December 28, 1905.

Figure 79*g* gives the half side elevation, two cross-sections, and plans of typical panels of the lateral systems for a highway bridge over the Mendota ravine, near St. Paul. The span of the arch is about 192 ft. and the extreme height about 95 ft. The chords are spliced with plain wooden fish plates at each panel point, and the diagonal struts bear against cast-iron angle blocks. The lower chords are reënforced by horizontal adjustable tension rods connected to them at the fourth panel point from the crown. This illustration is reproduced by permission from Types and Details of Bridge Construction; Part I, Arch Spans, by Frank W. Skinner.


4"x6" Stuck Rail
4"x4"
4"x4"
4"x4"
2" Plank
6"x8"
1" Bolts
6"x4"
4"x4"
3'6"
1¼" Rod
12'3"
6"x8"
12'6"
Middle Stringer
10'7"
8"x8" Cap
6"x8" Post
6"x8"
2" Splice Plank
8"x8" Sill
42'0"
6'0"
3'3"
1" Exp. Bolt
1'8"
40'4"
4'6"
5'0"
ENG. NEWS


4'0"
4'0"
2" Plank

Splice Plank.


1¼" Rod
4"x6" Truss
5'0"
1"x6" Iron
2" Plank
Splices
1" Exp. Bolt
8"x8" Sill
1" Exp. Bolts
6'0"

Cross Section at Center.

FIGS. 79*d*, *e* and *f*.—Framing of Ginn Field Foot Bridge, Metropolitan Park System.

Figure 79*d* is an excellent illustration of careful design in framing, to secure a simple structure of pleasing lines which is well adapted to its location. It crosses the Aberjina River at the upper end of Ginn Field, a portion of the Winchester section of the Mystic Valley Parkway. In type it is a combination of a King-post truss and an arch. The middle curved portions of the arch or floor stringers were built up of 2-inch planks for economic reasons. The timber used in the structure is all kyanized spruce of extra quality, cut to exact sizes and shapes at the mill before treatment. In assembling, the ends of all timber which butt together were given a good coat of lead, and the finished structure was given two coats of paint. A classified statement of the cost and a half-tone view of the bridge may be found in an article by W. T. Pierce, in *Eng. News*, vol. 54, page 265, Sept. 14, 1905.

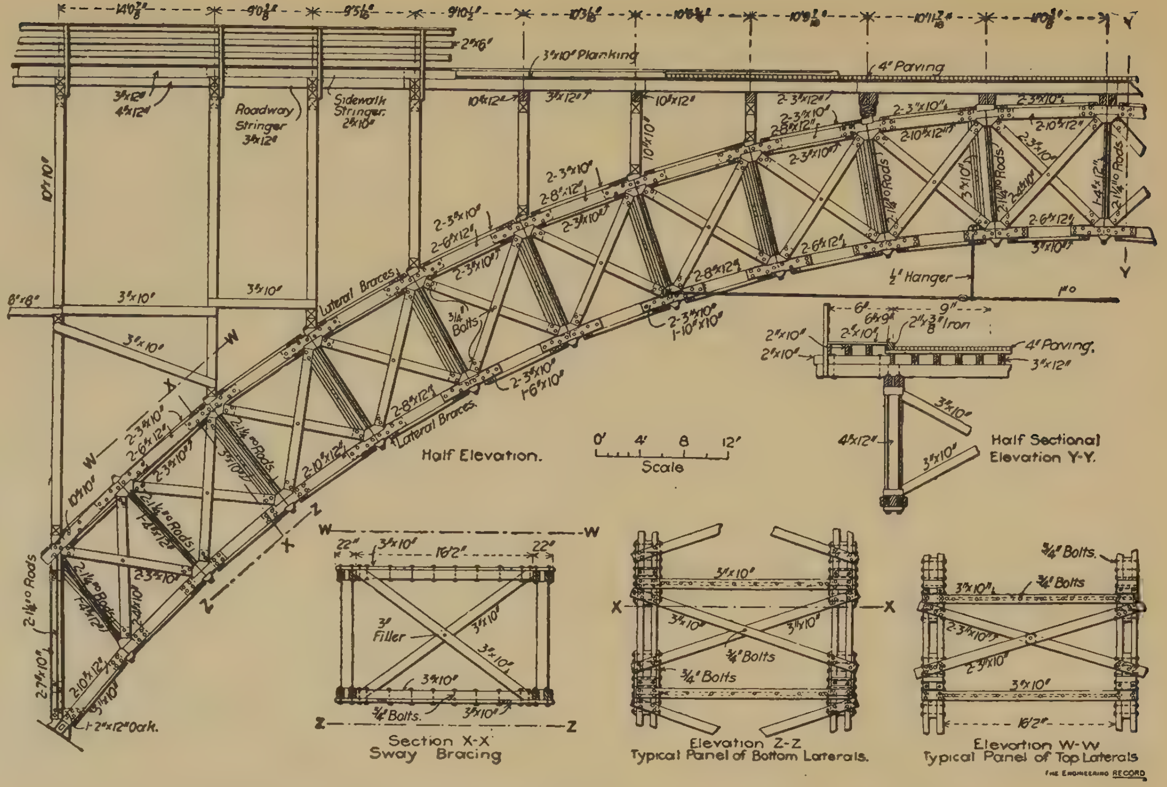

3"x10" Planking
4" Paving
Roadway Stringer 3"x12"
Sidewalk Stringer 2"x10"
10"x10"
8"x8"
Lateral Braces
Lateral Braces
2-1¼" Rods
2-3"x10"
2-8"x12"
2-6"x12"
2-10"x12"
3"x10"
½" Hanger
2"x3" Iron
4" Paving
3"x12"
4"x12"
1-2"x12" Oak.
Half Elevation.
Half Sectional Elevation Y-Y.
0' 4' 8 12'
Scale
¾" Bolts.
3" Filler
Section X-X
Sway Bracing
Elevation Z-Z
Typical Panel of Bottom Laterals.
Elevation W-W
Typical Panel of Top Laterals
THE ENGINEERING RECORD

FIG. 79*g*.—Wooden Trussed Arch Bridge over Mendota Ravine, near St. Paul, Minn.

ART. 80. ARCH CENTERING

Figures 80*a* to *f* show the general side elevation, central transverse sectional elevation, and certain enlarged details of the centers for the Piney Branch concrete arch bridge in Washington, D. C. The general arrangement of the posts and of some of the bracing differs materially from that of any centers previously constructed in this country, thus creating practically a new type. It is possible to compute with reasonable precision the stresses to which the various posts, struts, and beams are subject. The location of the temporary concrete footings and the varying inclination of the posts were so designed on account of the unique method of erection adopted, described and illustrated in Engineering News, vol. 55, page 453, April 19, 1906. Especial attention is called to the 6- by 6-in. struts between the 12- by 16-in. caps directly below the joists. The transverse bracing of each bent is remarkably simple and effective. The horizontal rails are all double and their ends are connected to the caps by the device illustrated in Fig. 80*f*. The tongued and grooved lagging is arranged by panels, as indicated in Fig. 80*c*, so as to facilitate the work of demolition. White-oak bearing blocks are placed on the footings, and above these the white-oak wedges which receive the vertical components of the thrusts of the posts, the horizontal components being resisted by the timbers into which the upper wedges are notched, as shown in Fig. 80*e*.

The gage rods shown in Figs. 80*a* and *d* were used for the purpose of keeping a record of the deformation of the centering during construction, and also of the deformation of the arch ring after construction. The movement of the bottom of each rod, with reference to a fixed elevation near the ground, is read and recorded from time to time so that all changes may be definitely known by the engineer. The deflections of the centers and of the arch ring, as observed by means of the gage rods, are shown graphically in an article published in Engineering News, vol. 57, page 682, June 20, 1907. The results give evidence of the vital importance of the plan adopted (see Fig. 80*b*) of making the distance between the exterior posts of each bent somewhat less than the total width of the bridge, so as to bring practically the same load on each post of a bent whether located at the face or in the middle. Such gage rods were first used by W. J. Douglas during

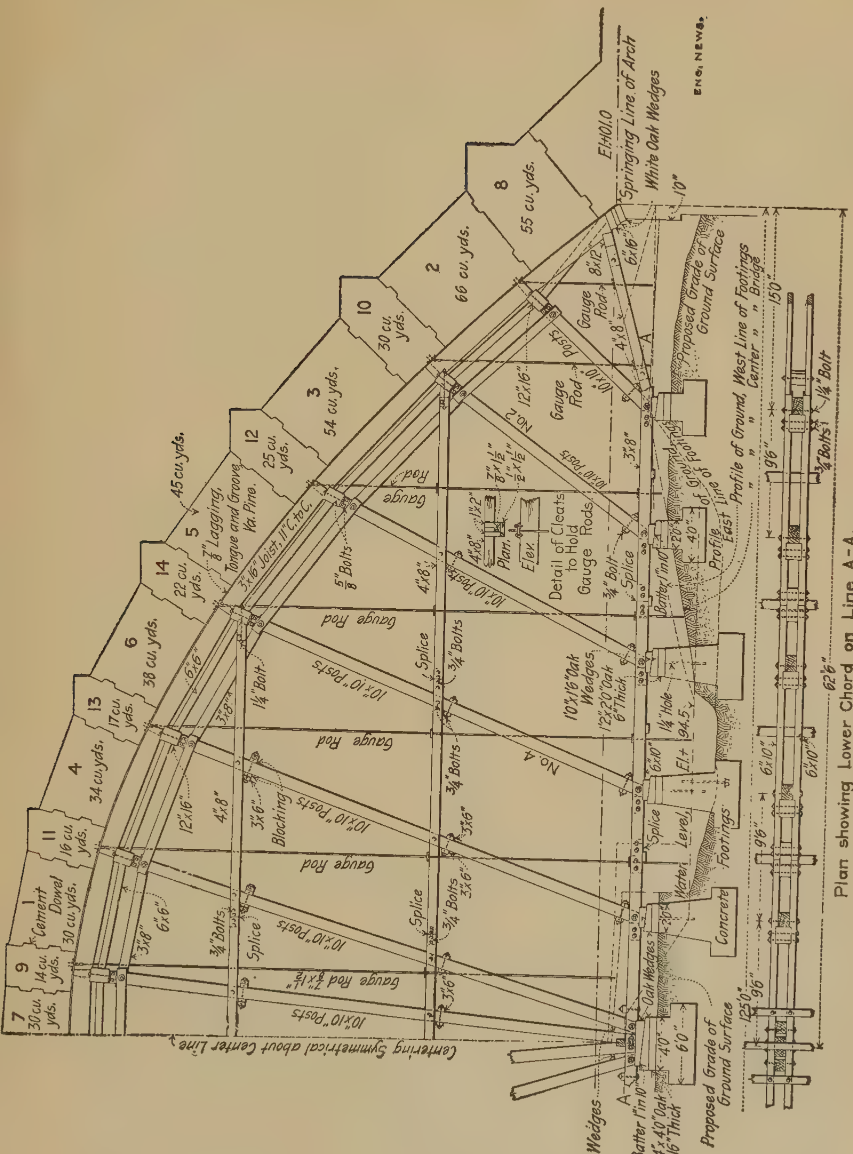
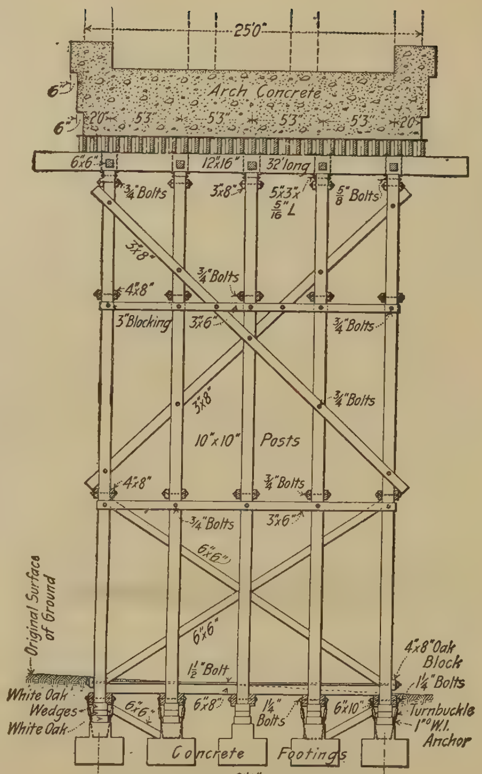
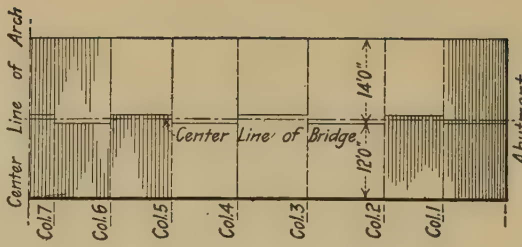

Plan showing Lower Chord on Line A-A,
Centering Symmetrical about Center Line
7 30 cu. yds.
9 14 cu. yds.
1 Cement Dowel 30 cu. yds.
11 16 cu. yds.
4 34 cu. yds.
13 17 cu. yds.
6 38 cu. yds.
14 22 cu. yds.
5 45 cu. yds.
12 25 cu. yds.
3 54 cu. yds.
10 30 cu. yds.
2 66 cu. yds.
8 55 cu. yds.
7/8" Lagging, Tongue and Groove Va. Pine.
3"x16" Joist, 11" C. to C.
El. 401.0
Springing Line of Arch
White Oak Wedges
1'0"
10"x10" Posts
Gauge Rod
Gauge Rod 7/8"x1 1/2"
3/4" Bolts
5/8" Bolts
1 1/4" Bolt
Splice
Blocking
12"x16"
4"x8"
3"x6"
3"x8"
6"x6"
6"x16"
8"x12"
No. 2
No. 4
A
Detail of Cleats to Hold Gauge Rods
Plan.
Elev.
7/8"x1 1/2"
1/2"x1 1/2"
1"x2"
3/4" Bolt
10"x16" Oak Wedges
12"x20" Oak 6" Thick
Batter 1" in 10"
1 1/4" Hole
El. 94.5
Water Level
6"x10"
Concrete Footings
Oak Wedges
Wedges
1/4"x4'0" Oak 16" Thick
4'0"
6'0"
2'0"
Proposed Grade of Ground Surface
Profile of Ground, West Line of Footings
" " " Center " "
Profile " " East Line of Bridge
15'0"
9'6"
12'6"
62'6"
6"x10"
1/4" Bolts
1 1/4" Bolt
ENG. NEWS.

FIG. 80a.—Centers for Piney Branch Concrete Arch Bridge on 16th Street, Washington, D. C.

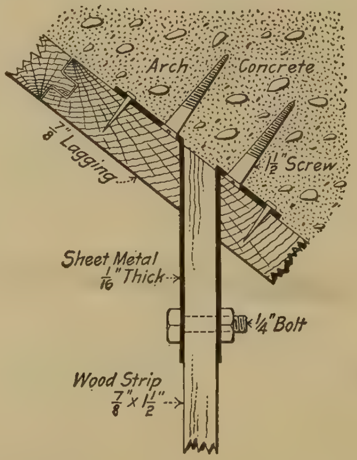

25'0"
6"
6"
Arch Concrete
2'0"
5'3"
5'3"
5'3"
5'3"
2'0"
6"x6"
12"x16"
32' long
3/4" Bolts
3"x8"
5"x3"x 5/16" L
5/8" Bolts
3"x8"
3/4" Bolts
4"x8"
3" Blocking
3"x6"
3/4" Bolts
3"x8"
3/4" Bolts
10"x10" Posts
4"x8"
3/4" Bolts
3/4" Bolts
3"x6"
6"x6"
6"x6"
1 1/2" Bolt
Original Surface of Ground
White Oak
Wedges
White Oak
6"x6"
6"x8"
1 1/4" Bolts
6"x10"
4"x8" Oak Block
1 1/4" Bolts
Turnbuckle
1" W.I. Anchor
Concrete Footings
21'0"

Transverse Section at Center.

FIG. 80b.


Center Line of Arch
Center Line of Bridge
14'0"
12'0"
Abutment
Col.7
Col.6
Col.5
Col.4
Col.3
Col.2
Col.1

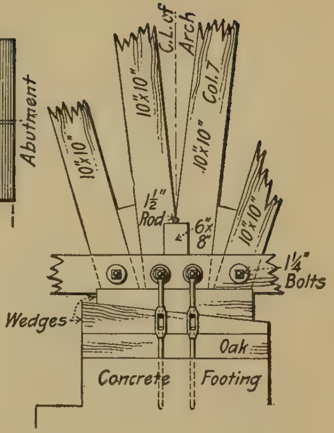
Layout of Lagging; Half Plan.


Arch Concrete
1 1/2" Screw
7/8" Lagging
Sheet Metal 1/16" Thick
1/4" Bolt
Wood Strip 7/8" x 1 1/2"

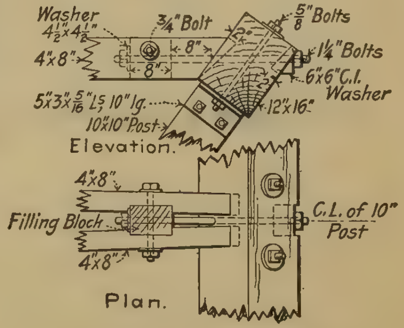
ENG. NEWS.

Detail of Gage Rod.

FIGS. 80c and d.


C.L. of Arch
10"x10"
10"x10"
Col.7
10"x10"
10"x10"
1/2" Rod
6"x8"
1 1/4" Bolts
Wedges
Oak
Concrete Footing

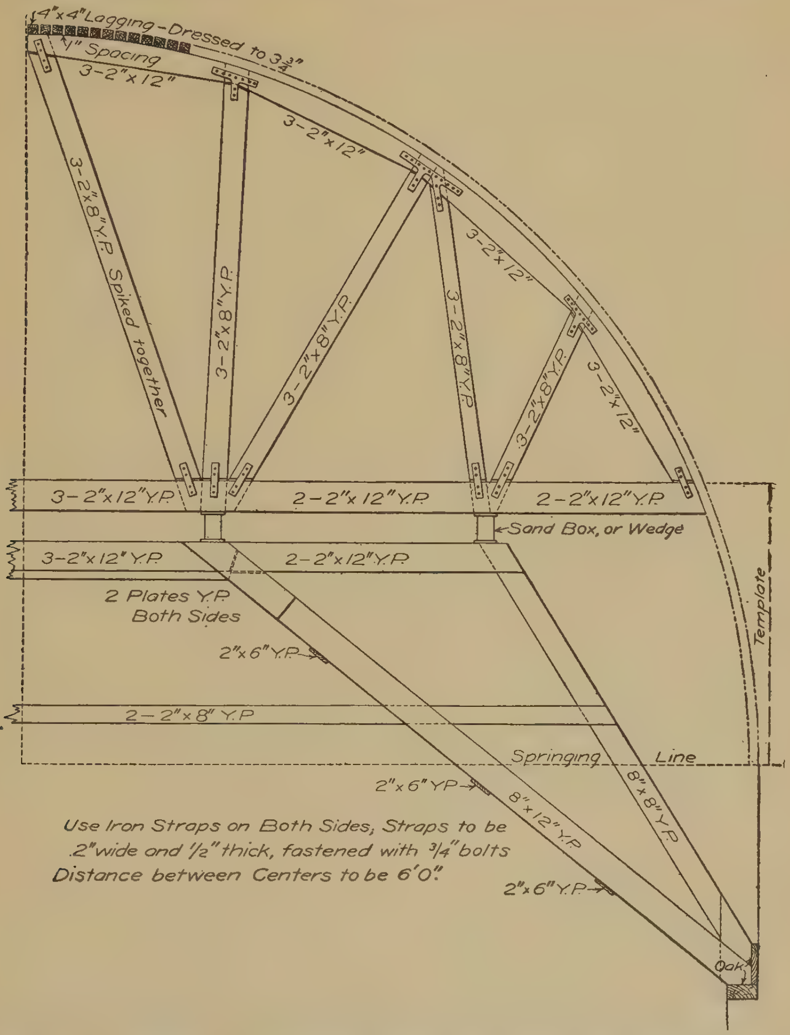
Base of Center Posts.


Washer 4 1/2"x4 1/2"
3/4" Bolt
5/8" Bolts
1 1/4" Bolts
6"x6" C.I. Washer
4"x8"
8"
8"
5"x3"x 5/16" L's 10" lg.
10"x10" Post
12"x16"
Elevation.
4"x8"
Filling Block
4"x8"
C.L. of 10" Post
Plan.

Detail at Cap of Columns 2 and 4.

FIGS. 80e and f.

the construction of the Connecticut Avenue concrete arch bridge in the same city.


4"x4" Lagging - Dressed to 3¾"
1" Spacing
3-2"x12"
3-2"x12"
3-2"x12"
3-2"x12"
3-2"x8" Y.P. Spiked together
3-2"x8" Y.P.
3-2"x8" Y.P.
3-2"x8" Y.P.
3-2"x8" Y.P.
3-2"x12" Y.P.
2-2"x12" Y.P.
2-2"x12" Y.P.
Sand Box, or Wedge
3-2"x12" Y.P.
2-2"x12" Y.P.
2 Plates Y.P. Both Sides
2"x6" Y.P.
2-2"x8" Y.P.
Template
Springing Line
2"x6" Y.P.
8"x12" Y.P.
8"x8" Y.P.
2"x6" Y.P.
Oak

Use Iron Straps on Both Sides; Straps to be 2" wide and ½" thick, fastened with ¾" bolts Distance between Centers to be 6'0".

Fig. 80*g*.—Framing of Centers for Stone Arch Bridge over Rocky Creek, on Massachusetts Ave., Washington, D. C.

The centers in Fig. 80*g* were used for a semicircular stone arch. The main horizontal timber is placed some distance above the springing line of the arch, thereby securing a more satisfactory web system for the upper frame. The lower frame acts somewhat like an arch. Straps and bolts are used as fastenings at most of the joints. Two other designs for centering by W. J. Douglas, and the modified construction as erected by the contractor, are published in Engineering Record, vol. 60, page 172, August 14, 1909.

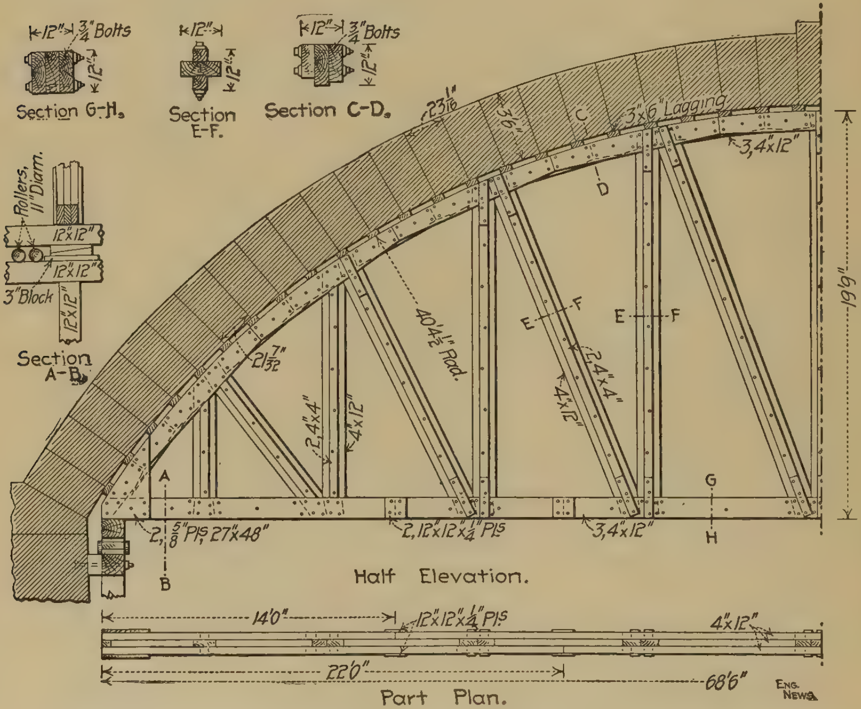

12" ¾" Bolts
12"
Section G-H.
12"
12"
Section E-F.
12" ¾" Bolts
12"
Section C-D.
Rollers, 1" Diam.
12"x12"
12"x12"
3" Block
12"x12"
Section A-B.
23 1/16"
36"
C
3"x6" Lagging
3,4"x12"
D
40' 4½" Rad.
E
F
E
F
2,4"x4"
4"x12"
21 7/32"
2,4"x4"
4"x12"
19'9"
A
G
2, 5/8" Pls 27"x48"
2,12"x12"x¼" Pls
3,4"x12"
B
H
Half Elevation.
14'0"
12"x12"x¼" Pls
4"x12"
22'0"
68'6"
Part Plan.
Eng. News

FIG. 80*h*.—Details of Centers for Rockville Bridge, Pennsylvania Railroad.

Figure 80*h* gives the details of the truss centers used in the construction of the Rockville arch bridge across the Susquehanna river on the Pennsylvania Railroad. Each center consisted of eight parallel trusses with a horizontal lower chord and a segmental upper chord built up of heavy plank. The special features include the cruciform section of the web members, the steel plate connections at the end joints, and the rollers adjacent to the wedge supports on the two-story pier bents, the sills of the bents resting on the projections of the concrete pier footings.

The center had only half the width of the arch ring, and after one-half of the ring was completed the center was lowered onto the rollers by knocking out the wedges and moved over into position for the construction of the second half. The bridge contains 48 spans, each 70 ft. long in the clear. Fig. 80*h* is reprinted from Engineering News, vol. 46, page 448, December 12, 1901.

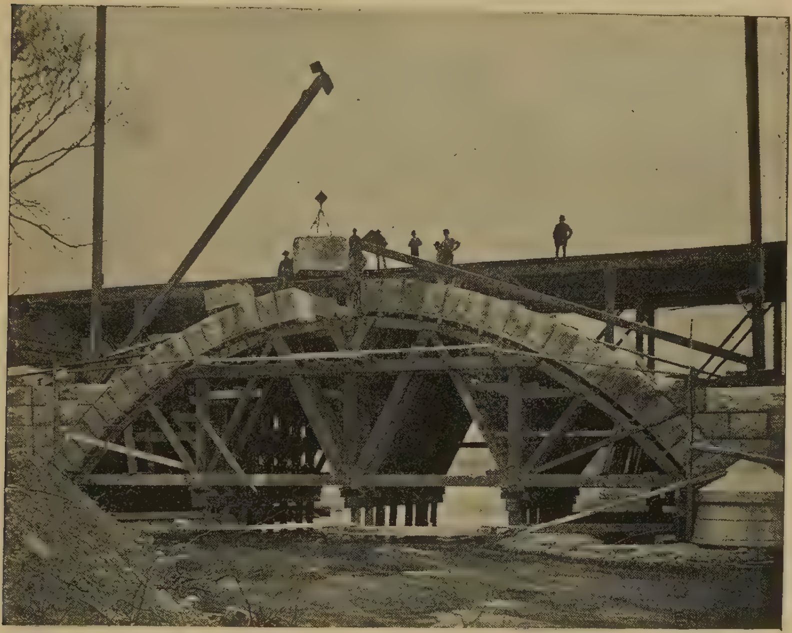


Fig. 80*i*.—Stone Arch Bridge of C., M. & St. P. Ry., Watertown, Wis.

The details of the center shown in Fig. 80*i* are given in Fig. 17*d*. It consists of 8 framed ribs spaced 3 ft. 10 in. apart, center to center, these all being supported on three intermediate bents of 8 piles each, and by blocking on each foundation footing. Lagging strips 4 by 5¾ in. were placed at each voussoir joint.

An interesting type of trussed center is the one used for the arches in the piers of the Queensboro cantilever bridge over the East River at Blackwell's Island, New York City, the span being 48 ft. in the clear. The lower chord or tie beam is 10 by 12 in., without a splice, the three long radial members consist of two pieces 3 by 10 in., the two long-chord and the four short-chord

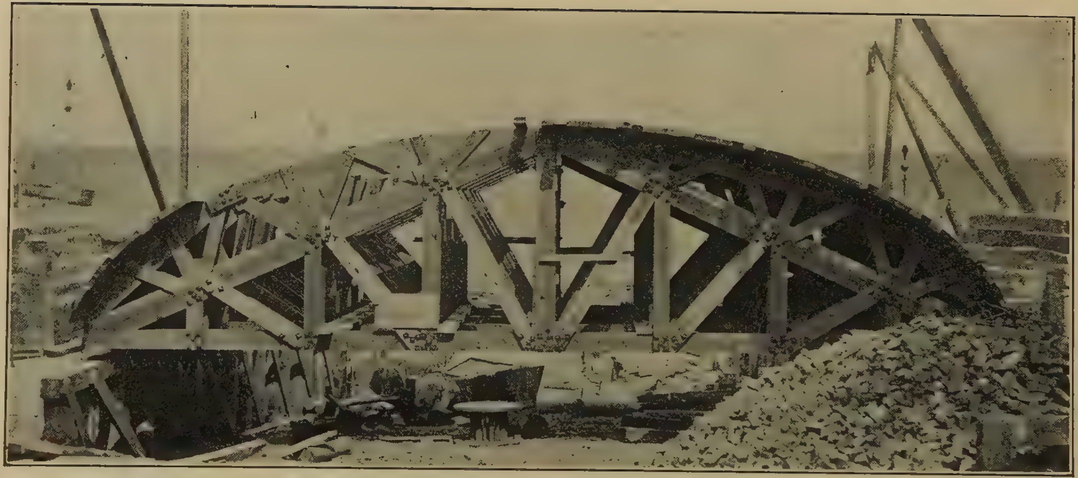


FIG. 80*j*.—Centers for an Elliptical Arch at 138th Street, Riverside Drive, New York City.

The centering is to support a concrete arch with granite faces, having a span of 50 feet and a rise of 12 feet 6 inches. The centers consist of triangular trusses having the same span, with braces radiating from the upper panel points to the scarf boards on which the lagging is laid. The connections at the panel points are made with bolts. An elevation of the arch reproduced from the architect's drawings and a half cross-section of the arch may be seen in *Eng. Record*, vol. 51, page 157, Feb. 11, 1905.

members are 10 by 12 in., the four short braces are composed of two pieces 3 by 10 in., and the filling pieces, above the short upper chord members, are 6 by 12, and 6 by 8 in. in section. Single and triple steps are used for the joints. The center rests on temporary stone corbels which are cut away after the arch is completed. See article on Construction of the Blackwell's Island Bridge Masonry, in Engineering Record, vol. 46, page 554, December 13, 1902. Additional references to centering are given in Art. 82.

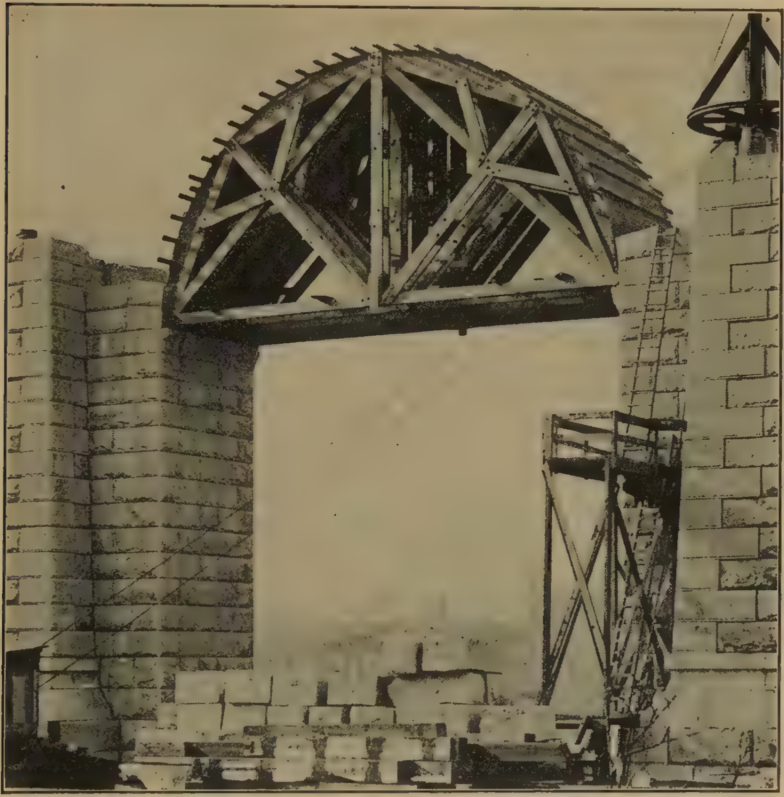


FIG. 80*k*.—Center for Pier Arch of Queensboro Bridge, New York.

ART. 81. MISCELLANEOUS STRUCTURES

The small Howe truss, with a span of 33 ft. 4 in., illustrated in Fig. 81*a*, was used over the main entrance to the Transportation Building of the World's Columbian Exposition at Chicago. The counter diagonals consist of two pieces with filler blocks between them at the ends, and are toe-nailed to the oak angle blocks. The main diagonals are held in position by dowels. Both chords consist of two sticks bolted together at intervals. Plaster joints are

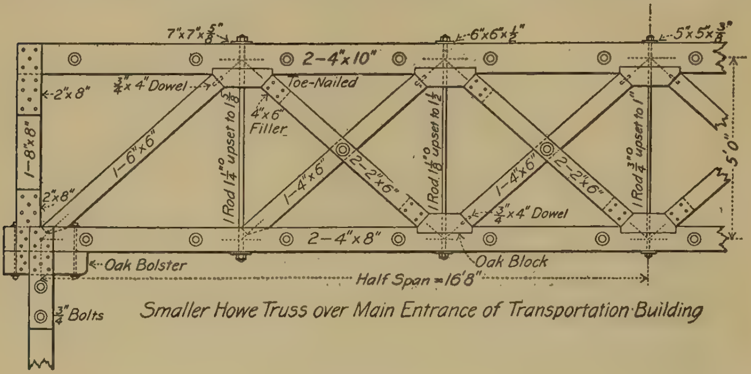

7"x7"x5/8"
6"x6"x1/2"
5"x5"x3/8"
2-4"x10"
Toe-Nailed
3/4"x4" Dowel
2"x8"
1-8"x8"
2"x8"
1-6"x6"
4"x6" Filler
1 Rod 1 1/4" upset to 1 1/2"
1-4"x6"
2-2"x6"
1 Rod 1 1/8" upset to 1 1/4"
1-4"x6"
2-2"x6"
1 Rod 3/4" upset to 1"
3/4"x4" Dowel
5'0"
2-4"x8"
Oak Bolster
Oak Block
Half Span = 16'8"
3/4" Bolts
Smaller Howe Truss over Main Entrance of Transportation Building

FIG. 81*a*.

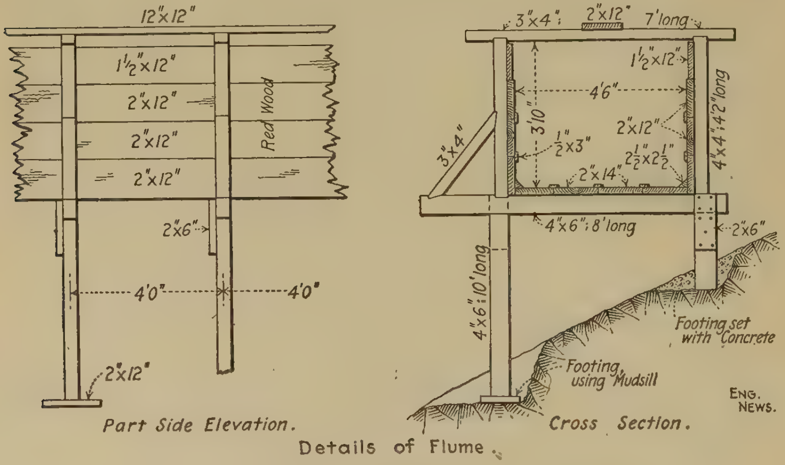

12"x12"
1½"x12"
2"x12"
2"x12"
2"x12"
Red Wood
2"x6"
4'0"
4'0"
2"x12"
Part Side Elevation.
3"x4"
2"x12" 7'long
1½"x12"
4'6"
3'10"
2"x12"
½"x3"
2½"x2½"
2"x14"
4"x4"; 4'2" long
3"x4"
4"x6"; 8' long
2"x6"
4"x6"; 10' long
Footing set with Concrete
Footing using Mudsill
Cross Section.
Eng. News.
Details of Flume.

FIG. 81*b*.—Flume on Dulzura Conduit, California.

used at the end panel points, and at the connection with the supporting posts.

Figures 81*b* and *c* give the typical details of a flume and supporting trestle on the line of the Dulzura conduit in Southern California. The trestle work is of pine, on concrete footing blocks, except in a few places where wooden mudsills are indicated in the drawing. The flume itself is of redwood. Battens cover the butt joints of the 2-in. plank, and triangular strips are put in the corners. The vertical posts of the frames are notched into sills and tie-timbers. The diagonal brace is stepped into the post and sill, and in Fig. 81*b* plaster joints are shown between the sill and its supporting posts.

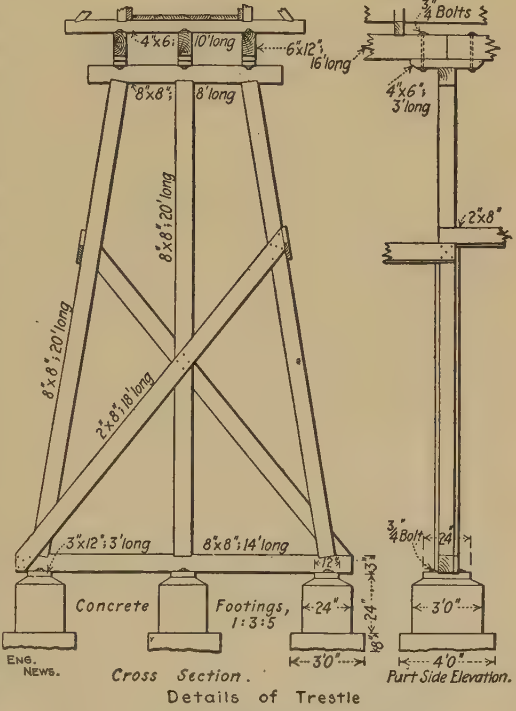

4"x6"; 10'long
6"x12"; 16'long
8"x8"; 8'long
8"x8"; 20'long
8"x8"; 20'long
2"x8"; 18'long
3"x12"; 3'long
8"x8"; 14'long
12"
3"
Concrete Footings, 1:3:5
24"
24"
18"
3'0"
Eng. News.
Cross Section.
Details of Trestle
3/4" Bolts
4"x6"; 3'long
2"x8"
3/4" Bolt
24"
3'0"
4'0"
Part Side Elevation.

FIG. 81*c*.—Trestle on Dulzura Conduit.

An interesting example of light framing, in which stiffness rather than strength is the most important consideration, is that of the temporary signals built by the Coast and Geodetic Survey of the United States, as described and illustrated in Appendix 4 to the Report of 1903, on Triangulation Southward along the Ninety-eighth Meridian in 1902, by John F. Hayford.

Oblique scarf joints are used for splicing the long posts of both tripod and scaffold which compose the signal, the length of the splice being about six times the thickness of the timber. For a height of 60 ft., 5- by 5-in. scantling is used for the posts and 2- by 4-in. scantling for the horizontal and diagonal braces. The

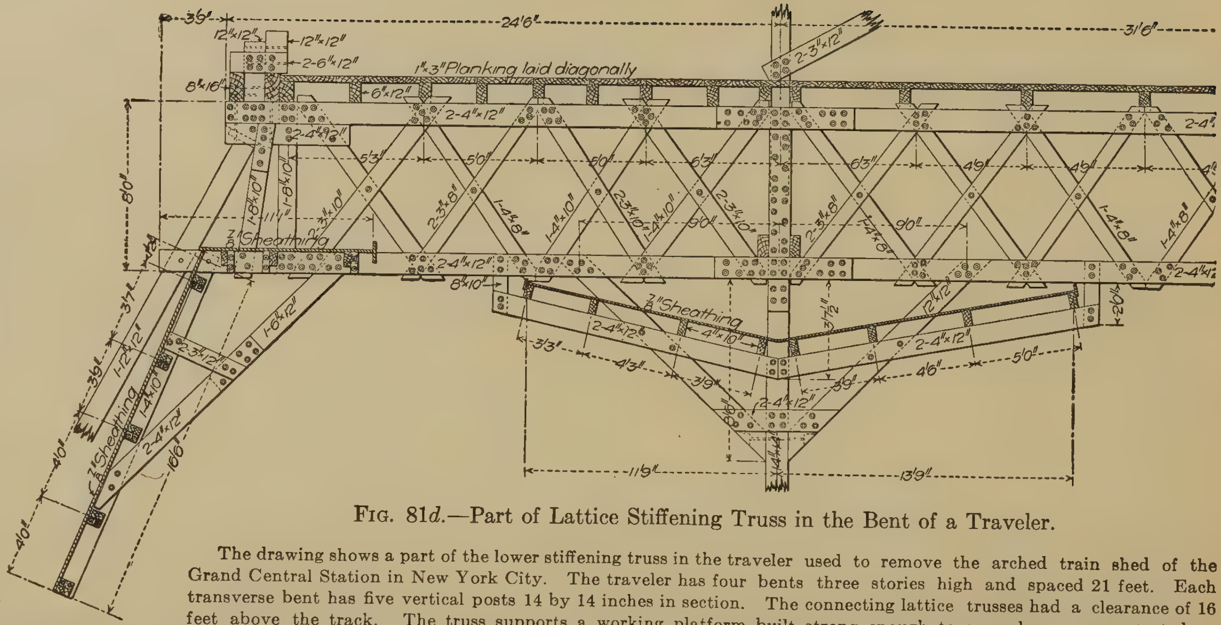

3'6"
24'6"
31'6"
12"×12"
12"×12"
2-6"×12"
8"×16"
1"×3" Planking laid diagonally
6"×12"
2-4"×12"
2-4"×12"
2-3"×12"
8'0"
5'3"
5'0"
5'0"
6'3"
6'3"
4'9"
4'9"
1-8"×10"
1-8"×10"
2-3"×10"
2-3"×8"
1-4"×8"
1-4"×10"
2-3"×10"
1-4"×10"
9'0"
2-3"×10"
2-3"×8"
1-4"×8"
9'0"
1-4"×8"
1-4"×8"
7/8" Sheathing
2-4"×12"
8"×10"
2-4"×12"
7/8" Sheathing
4"×10"
2-4"×12
2-4"×12"
2-4"×12"
3'7½"
2'0½"
3'3"
4'3"
3'9"
3'9"
4'6"
5'0"
11'9"
13'9"
1-6"×12"
2-3"×12"
1-12"×12"
1-4"×10
2-4"×12
7/8" Sheathing
3'7"
3'5"
4'0"
4'0"
16'6"
14"×14"

Fig. 81*d*.—Part of Lattice Stiffening Truss in the Bent of a Traveler.

The drawing shows a part of the lower stiffening truss in the traveler used to remove the arched train shed of the Grand Central Station in New York City. The traveler has four bents three stories high and spaced 21 feet. Each transverse bent has five vertical posts 14 by 14 inches in section. The connecting lattice trusses had a clearance of 16 feet above the track. The truss supports a working platform built strong enough to carry heavy concentrated or uniform loads. It is connected to each post by long knee braces, and supports sheathing over the platforms between the tracks. The end diagonal is a 12- by 12-inch timber extended to supports on the ground, forming inclined shores which are wedged up at the lower end when the traveler is in service. See *Eng. Record*, vol. 58, page 321, Sept. 19, 1908.

end of each stick is $\frac{3}{4}$ in. thick after the cut is made for the splice. The scarf joints are nailed with 60d nails, each brace whether horizontal or diagonal is fastened to each post by two 40d nails, and one 40d nail is used at each intersection of the diagonals. The extra stiffness required is secured by springing the posts before the braces are nailed into final position, thus subjecting the posts to initial flexure, and the horizontal braces to initial tension or compression. Additional details regarding the proportions of the tripod and scaffold and a description of the methods of erection and anchorage may be obtained by consulting the reference given above.

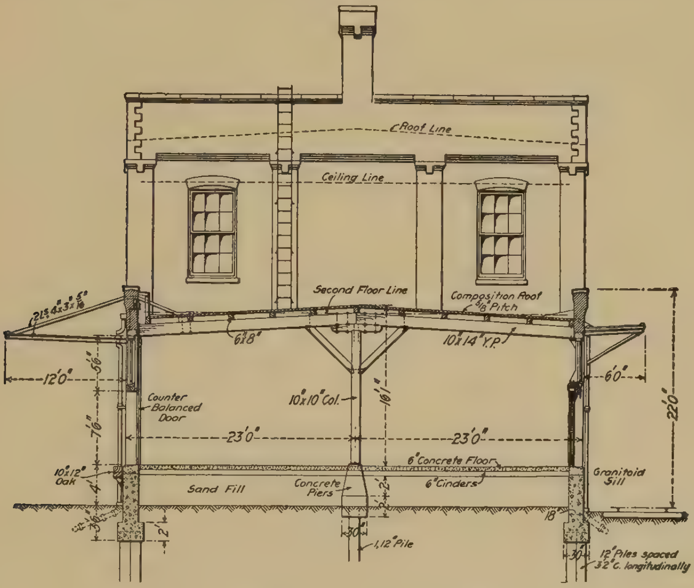
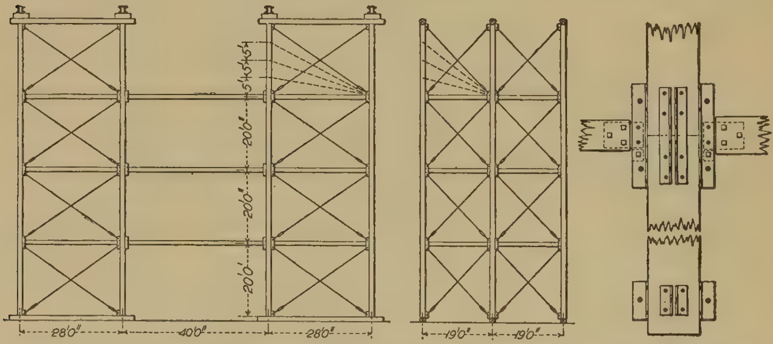

Roof Line
Ceiling Line
Second Floor Line
Composition Roof
3/8" Pitch
2 Ls 4"x3"x 5/16"
6"x8"
10"x14" Y.P.
12'0"
5'6"
7'6"
4'
3'6"
2'
10"x12" Oak
Counter Balanced Door
10"x10" Col.
16'1"
23'0"
23'0"
6'0"
22'0"
6" Concrete Floor
6" Cinders
Sand Fill
Concrete Piers
2'
2'
30"
1,12" Pile
18"
Granitoid Sill
30"
12" Piles spaced 32" c. longitudinally

Fig. 81*e*.—Cross-section through Freight House.

Figure 81*e* shows a cross-section of the combined terminal freight house at Galveston, Tex., of the Rock Island and the Colorado & Southern Railways. The freight house is 46 by 300 ft., with a platform at its west end 46 by 200 ft. The building has a second story for offices for 100 ft. of its length, the end elevation of which is also given in the illustration. It is built of brick, with composition roofing, and with concrete footings on a pile foundation. The inclined 10- by 14-in. yellow pine transverse beams support 6- by 8-in. purlins on which the sheeting is laid. The beams are connected to the supporting post by a keyed bolster and by angle braces having single step joints.

The falsework used in 1909 to erect the viaduct approaches of the Manhattan Bridge at New York City consists of wooden vertical posts and horizontal struts combined with adjustable steel diagonals. The connecting details at the joint are composed of steel angles, plates, pins, and bolts. As indicated in Fig. 81*f*, the lower three stories are of fixed height and an upper story of variable height, corresponding to the required elevation, and changing with the grade.

The connecting details are shown in the right-hand diagram, or Fig. 81*g*. The vertical posts consist of timber 14 in. square, have butt joints, and are spliced on each face by pairs of 3- by 2-in. vertical angles about 27 in. long, secured by four $\frac{3}{4}$-in. bolts through each angle. The angles of each pair are spaced 1 in. apart in the clear, to receive the vertical $\frac{1}{2}$-in. plates attached to the ends of its horizontal struts. The outstanding legs of the angles have holes for the $\frac{3}{4}$-in. bolts to connect them to these plates and angles, and also holes to receive the $1\frac{1}{4}$-in. pins, which pass through the loop eyes of the $\frac{7}{8}$-in. diagonal rods. The vertical plates are connected to the struts by insertion in slots at the ends of the timbers and fastened with three $\frac{3}{4}$-in. bolts.



FIGS. 81*f* and *g*.—Combination Falsework for Erection Viaduct Approaches.

The system of connections is simple and efficient, facilitates rapid assembling and dismantling without injury to the material, thus permitting it to be used over and over again, and secures considerable economy in handling as well as in the ultimate cost of the material, including freight charges. The connections are made symmetrical so that the corresponding members are interchangeable in the lower stories of fixed height. In case of uneven settlement under the posts the diagonals are adjusted so as to relieve undue stress. Accordingly the members can be accurately proportioned for the stresses required to support the known loads. The preceding facts and illustration are taken from an article on Progress on the Manhattan Bridge, in Engineering Record, vol. 60, page 116, July 31, 1909. See also an editorial on Combination Falsework on page 113.

The queen-post truss in Fig. 81*h* was designed by Ralph Modjeski to be used at each end of a pair of barges to hold them together during the construction of the pneumatic caissons for the Williamette river bridge at Portland, Ore., of the Northern Pacific Railway. Figure 81*i* gives a view taken September 8, 1906, of the caisson during construction, supported on beams which rest on the barges, and of the braced frames from which it is lowered to a floating position. It will be observed that the vertical members consist of two tie rods each, and which pass outside of the upper chord and take bearing on overhanging bent washers.

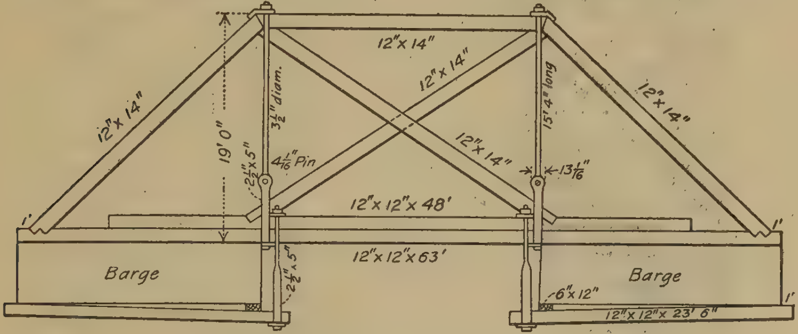


FIG. 81*h*.—Queen-post Truss.

On the sections of the Rapid Transit Subway extending from 60th to 104th Streets on Broadway, New York City, triangular wooden trusses with four panels were used to support the tracks for the surface cars during the excavation and construction for the subway. The view in Fig. 81*j* was taken at the corner of Broadway and 92nd Street. Transverse timbers spaced 5 ft. apart, extending under the conduit and track, were supported

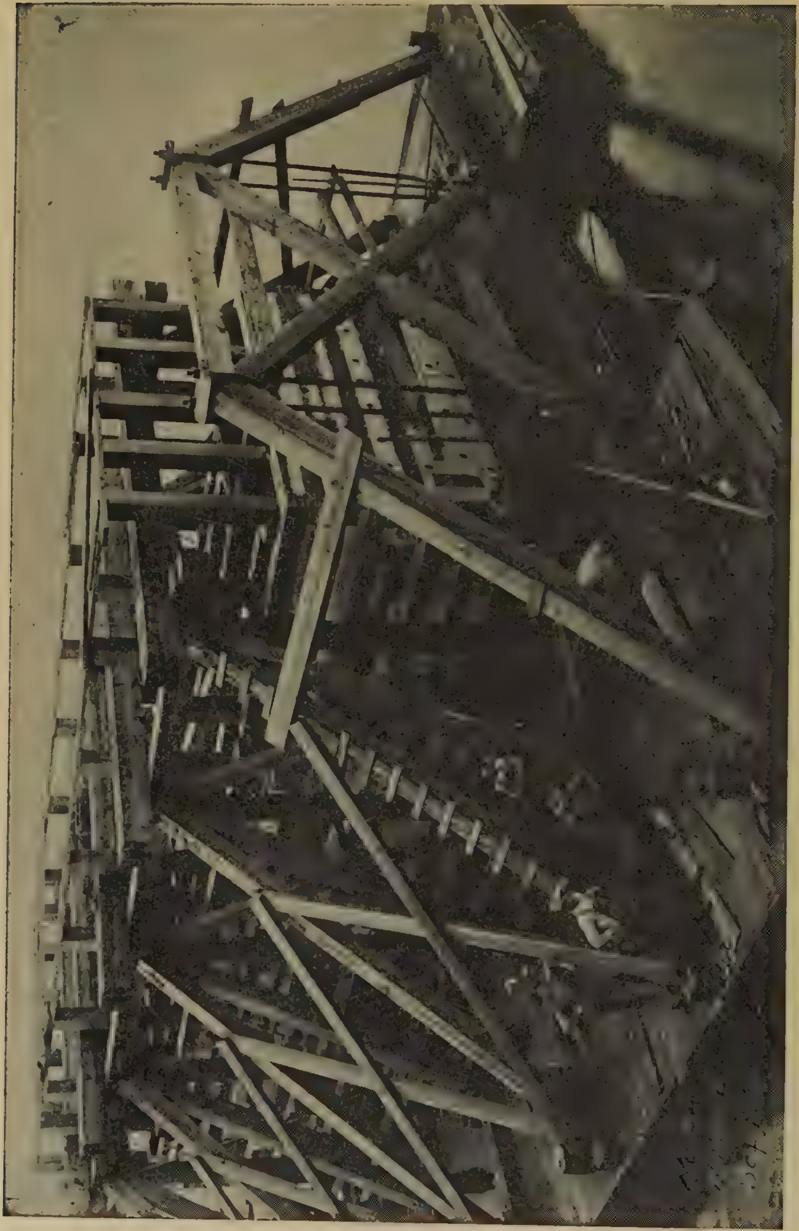


FIG. 81*i*.—Barges Supporting Pneumatic Caisson under Construction at Portland, Ore.

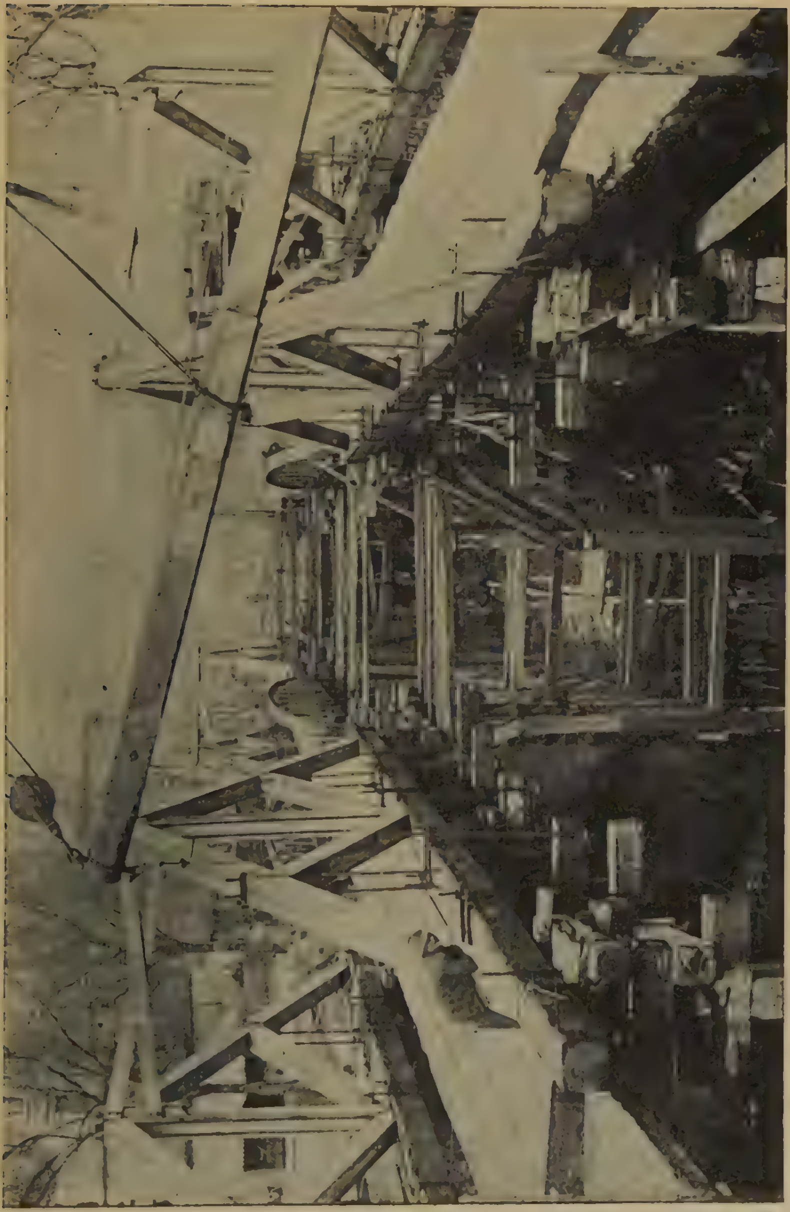


Fig. 81*j*.—Trusses Supporting Street Railway Tracks during Construction of New York Subway.

by bolt hangers from the lower chords of the trusses, having a span of nearly 60 ft. The panels next to the middle are $12\frac{1}{2}$ ft. long. The upper chords and diagonal struts are 12- by 12-in. timbers, and the lower chords are either 16 by 16 or 20 by 20 in. in section. The middle vertical consists of a 12- by 12-in. timber with two pairs of $1\frac{3}{4}$-in. round rods on each side, its upper end being connected to the upper chord timbers by two steel plates and bolt. The bearing plate on top is 12 by $\frac{3}{4}$ in. and 17 in. long. The side verticals consist of two $1\frac{3}{4}$-in. round rods each. The lower chord is subject to combined tension and flexure. The trusses are spaced 11 ft. apart in the clear, and braced together in the middle high enough to clear the street cars.

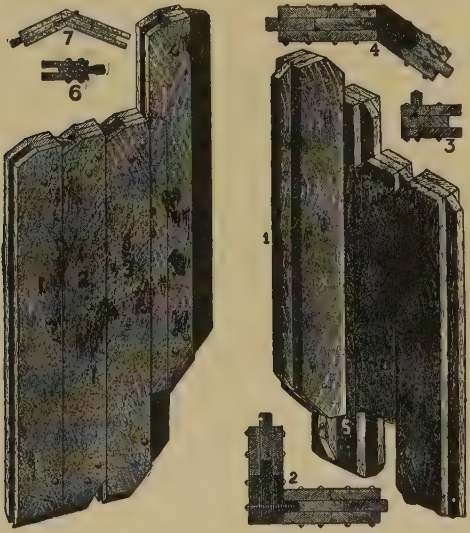
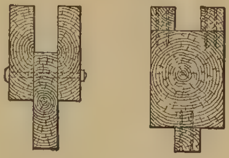


Fig. 81*k*.—Wakefield Sheet Piling.

An example of simple framing which has been very successfully employed since 1889 in the construction of sheet pile cofferdams consists essentially of three planks of equal width bolted and spiked together so as to form a tongue and groove (Fig. 81*k*). The center plank is sized to a regular thickness to insure a good fit between the tongues and grooves of adjacent piles. A $2\frac{1}{2}$-in. tongue is used for 1-in. boards, and a 4-in. tongue on 4-in. planks. The corresponding sizes of bolts are $\frac{3}{8}$ and $\frac{5}{8}$ in. in diameter. Two bolts are staggered in every 5 to 8 ft. of the length of the pile, spikes being used between the bolts on long piles.

These compound sheet piles have a much greater resistance to impact than a pile of the same gross cross-section composed of a single stick. The method of sharpening the pile in order to keep it in line, and to keep close to the adjacent one during driving, is shown in the illustration at the left. A view of a rectangular corner is shown at 1, and the sectional plan at 2, a tongue being bolted to the side of a pile when the corner is reached, as at 3 and 5. Any other angle is turned as indicated at 4 or 7. The relatively long tongue and groove is indicated at 6. Figures 81*l* and *m* show a section of a large pile in which the tongue and groove are formed by spiking separate pieces to the large timber.



FIGS. 81*l* and *m*.—Sections of Sheet Piles.

ART. 82. REFERENCES TO ENGINEERING LITERATURE

The following selected references relate to the subjects of the preceding articles in this chapter. As indicated in Art. 74, their purpose is merely to illustrate methods and details of framing, and to serve as an introduction for further study by the student. It will be a valuable exercise to supplement these lists by references to other periodicals arranged in card catalogue form.

SLOW-BURNING CONSTRUCTION

Mills and Mill Engineering, by Edward Sawyer, with discussion by John R. Freeman and others.—Eng. Rec., v. 21, p. 148, March 22, 1890.

Lessons of the Park Place Disaster, by Edward Atkinson.—Eng. Mag., v. 2, p. 137, Nov. 1891.

Some Notes on Fire Protection Engineering, by John R. Freeman.—Trans. Assoc., C. E. Cornell, 1894, v. 2, p. 85.

Our Enormous Annual Loss by Fire; A Waste Born of Ignorance and Neglect, by Edward Atkinson.—Eng. Mag., v. 7, p. 603, Aug. 1894.

Construction of Slow-burning Buildings, by Francis C. Moore.—Eng. Mag., v. 14, p. 955, March, 1898.

Designs of Factory Buildings, by Hales & Ballinger.—Eng. Rec., v. 44, p. 505, Nov. 23, 1901.

Comparative Cost of Wood and Steel Frame Factory Buildings, by H. G. Tyrrell.—R. R. Gaz., v. 37, p. 437, Oct. 4, 1904.

Improved Method of Saw-tooth Roof Construction, by Samuel M. Green.—Eng. News, v. 60, p. 263, Sept. 3, 1908; Eng. Rec., v. 55, p. 53, Jan. 12, 1907.

What is Mill Building Construction, by F. W. Dean.—Eng. News.-Rec., v. 79, p. 1184, Dec. 27, 1917.

Heavy Timber Mill Building Construction, by National Lumber Manufacturers Assoc.—Engineering Bulletin, No. 2, May, 1916.

TRESTLE CONSTRUCTION

Design of Wooden Trestles.—Editorial in Eng. News, v. 20, p. 50, July 21, 1888.

Use of Wood in Railroad Structures, Chap. XIX. Details of Trestle Construction, by Charles D. Jameson.—R. R. and Eng. Jour., v. 4, p. 21, January, 1890.

Construction of the St. Louis, Peoria & Northern Railway, by F. G. Jonah.—Eng. News, v. 43, p. 110, Feb. 15, 1890.

Rocky Mountain Work on the Great Northern, by James H. Kennedy.—R. R. Gaz., v. 26, p. 696, Oct. 12, 1894.

Trestle for Single Track Electric Railway.—Eng. News, v. 35, p. 255 (inset), April 16, 1896.

American Railway Bridges and Buildings, by Walter G. Berg. 1898, pp. 9–11, 198, 206, 207, and 436.

Reconstruction of the Utah Central Railway, by W. P. Hardesty.—Eng. News, v. 45, p. 44, Jan. 17, 1901.

Railway Trestle with Bents of Reinforced Concrete, by W. A. Allen.—Eng. News, v. 49, p. 244, March 12, 1903.

High Timber Trestle on the North Alabama Railroad.—Eng. Rec., v. 50, p. 110, July 23, 1904.

Building of the Chicago, Cincinnati & Louisville, and Cincinnati, Richmond & Muncie Railroads, by H. L. Weber and Fred R. Charles.—Eng. Rec., v. 51, p. 64, Jan. 21, 1905.

Standard Plans for Pile and Timber Trestle Bridges; Santa Fe Railway System, by A. F. Robinson.—Proc. Am. Ry. Eng. & M. of W. Assoc., 1905, v. 6, p. 43.

Wooden Trestle Bridges with Ballast Floors.—Proc. Am. Ry. Eng. M. W. Assoc., 1908, v. 9, p. 320; 1906, v. 7, p. 705.

Standard Trestle Bridge; Denver, Northwestern & Pacific.—R. R. Gaz., v. 40, p. 503, May 18, 1906.

Hints on the Design and Construction of Wooden Trestles, by R. Balfour.—Eng. News, v. 57, p. 610, June 6, 1907.

Design and Construction of Automatic Car Systems, by Charles J. Steffens.—Eng. News, v. 59, p. 165, Feb. 13, 1908.

Large Wooden Trestle at McGill, Nevada, by L. J. Dobbins.—Eng. News v. 59, p. 409, April 16, 1908.

Large Timber Trestles on the Pacific Coast Extension of the Chicago, Milwaukee & St. Paul Railway.—Eng. News, v. 61, p. 307, March 25, 1909.

Protection of Members on Wooden Bridge Structures.—Eng. Rec., v. 59 p. 637, May 15, 1909.

New Railway in Louisiana Cuts Off Long Ferry Transfer, by P. F. Lyons.—Eng. News-Rec., v. 99, p. 540, Oct. 6, 1927.

Creosoted Timber Trestles for Louisiana Highway.—Eng. News-Rec., v. 97, p. 918, Dec. 2, 1926.

Missouri Builds Timber Trestles with Concrete Floor.—Eng. News-Rec. v. 96, p. 536, April 1, 1926.

TIMBER BRIDGES

Queen-post Truss. Trestle for Single-track Electric Railway.—Eng. News, v. 35, p. 255 (inset), April 16, 1896.

Erection of the Park Avenue Viaduct, New York: Part 1, Falsework Supporting Temporary Tracks and New Center Girders.—Eng. Rec., v. 35, p. 269, Feb. 27, 1897.

Ornamental Bridges in a Private Park.—Eng. Rec., v. 47, p. 556, May 23, 1903.

Trestle Approaches and Skew Spans in Bridge Ten near Liberal Arts Building.—Eng. Rec., v. 49, p. 579, May 7, 1904.

Design of Timber Howe Trusses, by R. Balfour.—Eng. News, v. 58, p. 224, Aug. 29, 1907.

Weehawken Transfer Bridges of the West Shore Railroad.—Eng. Rec., v. 57, p. 185, Feb. 15, 1908.

Pontoon or Floating Drawbridges.—Eng. News, v. 59, p. 474, April 30, 1908.

Highway Bridge across the Kansas River at Fort Riley, Kansas, by P. S. Pond.—Eng. Rec., v. 58, pp. 44, 75, July 11, 18, 1908.

[Standard Design for Typical Wooden Overhead Highway Bridge, Single Track Crossing], Carolina, Clinchfield & Ohio Railway, by G. S. Fowler.—R. R. Age Gaz., v. 46, p. 539, March 19, 1909.

Timber Bridge Problems on Alaska Ry.—Eng. News-Rec., v. 83, p. 420, Aug. 28, 1919.

ARCH CENTERING

Arch Centers, by James W. Rollins, Jr.—Jour. Assoc. Eng. Soc., v. 27, p. 9, July, 1901.

Melan Arch Park Bridges at Washington, D. C., by W. J. Douglas.—Eng. News, v. 46, p. 323 (inset), Oct. 31, 1901.

Concrete-steel Bridge at Plano, Ill., Chicago, Burlington & Quincy Railway.—Eng. News, v. 52, p. 559, Dec. 22, 1904; Eng. Rec., v. 49, p. 18, Jan. 2, 1904; R. R. Gaz., v. 35, p. 886, Dec. 11, 1903.

Stone Arch Bridge on the Chicago, Milwaukee & St. Paul Railway at Watertown, Wis.—Eng. News. v. 49, p. 266, March 26, 1903 (see Figs. 17*d* and 80*i*).

Improvements on the Big Four Line between Lawrenceburg Junction and Sunman.—Eng. Rec., v. 49, p. 292, March 5, 1904.

Structural Details of the New Reinforced Concrete Bridge at Grand Rapids, Mich., by W. F. Tubesing.—Eng. News, v. 52, p. 489, Dec. 1, 1904.

Connecticut Avenue Concrete Arch Bridge, Washington, D. C.—Eng. Rec., v. 52, p. 30, July 8, 1905.

Concrete Viaducts of the Thebes Bridge.—Eng. Rec., v. 52, p. 128, July 29, 1905.

Reinforced Concrete Double-track Railroad Arch Bridge.—Eng. Rec., v. 52, p. 295, Sept. 9, 1905.

Danville Arch Bridge of the Cleveland, Cincinnati, Chicago & St. Louis Railway.—Eng. Rec., v. 53, p. 238, March 3, 1906.

Arch Rib Bridge of Reinforced Concrete at Grand Rapids, Mich., by George Jacob Davis.—Eng. News, v. 55, p. 321, March 22, 1906.

Concrete Arch on the Big Four at Danville.—R. R. Gaz., v. 41, p. 30, July 13, 1906.

Special Form of Arch Centering, by J. H. Milburn.—Eng. News, v. 56, p. 207, Aug. 23, 1906.

Construction of the 175th Street Arch, New York City.—Eng. Rec., v. 56, p. 379, Oct. 5, 1907.

Design of the Centering for the 233-Foot Arch Span, Walnut Lane Bridge, Philadelphia, Pa., by George Maurice Heller.—Proc. Engrs. Club of Philadelphia, v. 25, p. 257, July, 1908.

Wyoming Avenue Arch Bridge at Philadelphia.—Eng. Rec., v. 59, p. 233, Feb. 27, 1909.

Construction of the Edmondson Avenue Bridge, Baltimore.—Eng. Rec., v. 60, p. 172, Aug. 14, 1909.

Miscellaneous Structures

Memphis Bridge by George S. Morison, 1894.—Plates 11, 12, 13, 14, 16, 17, 19, 20, 21.

Falsework for Erecting the Manhattan Towers and End Spans; New East River Bridge, by C. E. Fowler.—Eng. News, v. 43, p. 164 (inset), March 8, 1900.

Erection of the Northwest Miramichi Bridge, Newcastle, N. B., by H. D. Bush.—R. R. Gaz., v. 35, p. 58, Jan. 23, 1903.

Building of a World's Fair, How the Louisiana Purchase Exposition has been Constructed.—Eng. Rec., v. 49, p. 568, May 7, 1904.

Building Construction of the St. Louis Exhibition.—Eng. News, v. 51, p. 478, May 19, 1904.

Erection of the 232-Foot Triangular Span of the Fraser River Bridge.—Eng. Rec., v. 56, p. 192, Aug. 13, 1904.

Baltimore Terminal of the Western Maryland R. R.—Eng. Rec., v. 52, p. 4, July 1, 1905.

Steamship Terminal with Concrete Pile Piers at Brunswick, Ga.; Atlantic & Birmingham Railway.—Eng. News, v. 56, p. 654, Dec. 20, 1906.

Floating Derrick for Handling Heavy Rip-rap Stone.—Eng. News, v. 57, p. 560, May 23, 1907.

Problem in Underpinning.—Eng. Rec., v. 56, p. 94, July 27, 1907.

[Details of 32-Foot Span Timber Flume.] Kern River No. 1 Power Plant of the Edison Electric Co., Los Angeles, by C. W. Whitney.—Eng. Rec., v. 56, p. 140, Aug. 10, 1907.

Roundhouse of the Lehigh and Hudson River Railway at Warwick, N. Y.—Eng. Rec., v. 57, p. 150, Feb. 8, 1908.

[Suspended Needlebeams for Underpinning the Schermerhorn Building.] Development of Building Foundations, by Frank W. Skinner.—Eng. Rec., v. 57, p. 412, April 4, 1908.

Closure of the Charles River Dam, by Edward C. Sherman.—Eng. News, v. 60, p. 498, Nov. 5, 1908.

Municipal Ferry House Substructure, New York.—Eng. Rec., v. 58, p. 525, Nov. 7, 1908.

Traveler with Pin-connected Braces, Mercantile Bridge across the Monongahela River.—Eng. Rec., v. 58, p. 686, Dec. 19, 1908.

Some Problems in Designing the Kern River No. 1 Hydro-electric Power Plant, by F. C. Finkle.—Eng. News, v. 60, p. 701, Dec. 24, 1908.

Gravel Washing Plant on the Richmond, Fredericksburg & Potomac R. R., by E. M. Hastings.—Eng. News, v. 61, p. 406, April 15, 1909.

Center and End Towers, Duplex Cableway.—Eng. Rec., v. 67, p. 487, May 3, 1913.

Motor Race Track at Chicago.—Eng. News, v. 74, p. 488, Sept. 9, 1915.

Extensive Great Northern Snow Shed Construction.—Ry. Age Gaz., v. 56, p. 902, April 17, 1914.

Safety Scaffolds for Buildings.—Eng. News, v. 75, p. 375, Feb. 24, 1916.

Details of Heavy Timber Framing.—Eng. News.-Rec., v. 84, p. 1016, May 20, 1920.

Timbering of Trenches, Shafts and Tunnels.—Eng. and Contr., v. 65, p. 143, September, 1926.

CHAPTER VI

COMMERCIAL GRADES, TIMBER TESTS AND UNIT STRESSES

Art. 83. Commercial Grades and Sizes

According to rules of the American Railway Engineering Association (1929), lumber is classified according to use as: (a) yard lumber, (b) structural timbers, and (c) shop or factory lumber. Different grading rules apply to each class of lumber. Yard lumber is lumber that is less than 5 in. in thickness and is intended for general building purposes. The grading of yard lumber is based upon the use of the entire piece. Structural timber is lumber that is 5 in. or over in thickness and width. The grading of structural timbers is based upon the strength of the piece and the use of the entire piece. Shop or factory lumber is lumber intended to be cut up for use in further manufacture.

Yard lumber is classified as strips, boards, and dimension, the latter being further subdivided into planks, scantlings and heavy joists. These divisions are based on size as follows: Strips, less than 2 in. thick and under 8 in. wide; boards, less than 2 in. thick and 8 in. or over in width; planks, from 2 to 4 in. thick and 8 in. and over in width; scantlings, 2 in. to 5 in. thick and under 8 in. in width; and heavy joists, 4 in. thick and 8 in. or more in width.

Another classification of lumber is according to its manufacture, namely, rough, surfaced, or worked. Rough lumber is undressed material as it comes from the saw. Surfaced lumber is material that is dressed by being run through a planer. It may be surfaced on one side (S1S), on two sides (S2S), on one edge (S1E), on two edges (S2E), or a combination of sides and edges, (S1S1E), (S2S1E), (S2S2E) or (S4S).

Worked lumber is material that has been run through a matching machine, sticker or molder. It may be matched, shiplapped or patterned. Matched lumber is material that is dressed and shaped to make a close tongue and groove joint at the edges or ends when laid edge to edge or end to end. Shiplapped lumber is material that is edge dressed to make a close rabbetted or lapped joint when laid edge to edge. Patterned lumber is worked material that is shaped to a patterned or molded form.

Softwood dimension and timbers where working stresses are required are classified as joists and planks, beams and stringers, or posts and timbers. The nominal dimensions are: joists and planks, 2 to 4 in. thick and 4 in. or more wide; beams and stringers 5 in. or more thick and 8 in. or more wide; and posts and timbers, 6 by 6 in. and larger. Rough structural material shall not be thinner or narrower than the nominal dimensions less $\frac{1}{4}$ in. for sizes 7 in. and under, and less $\frac{3}{8}$ in. for sizes 8 in. and over. Dressed material may be $\frac{1}{8}$ in. less than rough material.

The standard yard 1-in. board has a $\frac{25}{32}$-in. dressed thickness, and the extra standard has $\frac{1}{32}$ in. more thickness. Standard dimension 2-in. yard lumber has a dressed thickness of $1\frac{5}{8}$ in., and extra standard is $\frac{1}{8}$ in. more.

This reduction in size of the dressed dimensions over the nominal dimensions involves a large reduction in strength and stiffness for the smaller sizes. For example, a 2- by 6-in. joist worked down to $1\frac{5}{8}$ by $5\frac{5}{8}$-in. dimensions has a strength and stiffness of about 71 and 67 percent, respectively, of joists measuring exactly 2 by 6 in.

The standard lengths in the United States for structural material are multiples of 2 ft., whereas standard commercial sections are full inches up and including 4 in. and even inches beyond. For example, a piece 3 in. by 3 in. by 18 ft. is standard, but a piece 5 in. by 12 in. by 18 ft. is not.

Softwood structural timber is divided into four grades; dense-select (Douglas fir and southern pine), select (softwood species except southern pine), dense common (Douglas fir and southern pine), and common (all softwood species). The rules regarding the quality of lumber, the definitions of defects, and to what extent defects are allowed, are known as grading rules. An historical account of the development of grading in this country, together with copies of the grading rules, was published in 1906 in Bulletin 71 of the U. S. Forest Service, entitled Rules and Specifications for the Grading of Lumber Adopted by the Various Lumber Manufacturing Associations of the United States, compiled by E. R. Hodson. For up-to-date grading rules the reader is referred to Circular 295 of the U. S. Department of Agriculture, entitled Basic Grading Rules and Working Stresses for Structural Timbers (1923), by J. A. Newlin and R. P. A. Johnson, to U. S. Department of Commerce Bulletin 16, entitled Lumber (1926) and to the 1927 and 1929 Proceedings of the American Railway Engineering Association and of the American Society for Testing Materials.

For descriptions and illustrations of American woods used for structural work consult a book on Wood and Other Organic Structural Materials, by C. H. Snow. Other valuable books on the mechanical properties of timber are: Properties and Uses of Wood, by Arthur Koehler; Timber—Its Strength, Seasoning and Grading, by H. S. Betts; and Mechanical Properties of Wood, by S. J. Record. For further information see Bulletin 556 of the Forest Service, 1917, on Mechanical Properties of Woods Grown in the United States, by J. A. Newlin and T. R. C. Wilson; Bulletin 10, 1903, on Timber: An Elementary Discussion of the Characteristics and Properties of Wood, by Filibert Roth; and Bulletin 41, 1903, on Seasoning of Timber, by Hermann Von Shrenk. See also Wood Construction, by D. F. Holtman.

Art. 84. Factors Influencing Strength of Timber

Among the factors influencing the strength of timber are density, moisture content, size, and presence of defects. As the specific gravity of wood substance is practically constant, density measures the amount of wood substance in a given volume and as strength depends on the amount of wood substance, the denser the wood the greater is its strength. For this reason pieces of exceptionally light weight are not permitted in the higher grades of structural timber.

In southern pine and Douglas fir the proportion of summerwood, the dark portion of the annular ring, furnishes a practical means of estimating density. The contrast in color between summerwood and springwood should be distinct. The rate of growth of these species is not as great an assurance of strength as percentage of summerwood; but, for the better grades, material having not less than six annular rings per inch is required.

The direct effect of loss of moisture in seasoning green timber is to increase the strength by stiffening and strengthening the wood fibers. On the other hand this loss of moisture is accompanied by checking, splitting, warping and twisting, the amount depending on the size of the piece and the method of drying, and as a result some of the strength due to drying is lost. Timbers are also subject, during use, to varying conditions of moisture, from the dry condition of a heated building to the wet condition of a dock.

The size factor influencing strength is due to the fact that, as above stated, the larger the piece the more pronounced are the defects due to seasoning. In general, material 4 in. and less in thickness can carry higher unit stresses than material over 4 in. thick.

Any irregularity occurring in or on a piece of timber that may lower its strength is called a defect. Among the defects and blemishes that are recognized in the lumber trade are checks, shakes, cross breaks, "compression failure," cross-grain, knots, decay, splits, warp, holes, bark pocket, gum spots and streaks, pitch, pith, stain, birdseye, wane, and imperfect manufacture.

A *check* is a lengthwise separation of the wood, which usually occurs across the rings of annual growth, being due largely to the drying out of the wood from the outside toward the center. This drying is accompanied by shrinkage and it is this greater shrinkage of the outside as compared with the center that causes cracking.

A *shake* is a lengthwise separation of the wood, which usually occurs between and parallel to the rings of annual growth. Shakes will not result from drying unless there is a weakness in the bond between the rings at the time of cutting. It was formerly thought that shakes were caused by wind but there is little evidence to prove this.

Cross breaks are tension failures usually due to a local abnormal longitudinal shrinkage. Wood having this characteristic is called "proud" or "compression" wood. In fir and pine "compression" wood usually shows an absence of sharp contrast between springwood and summerwood.

A *compression failure* is a defect occurring as a result of excessive stress due to wind action on the living tree, the falling of the tree, or rough handling of logs or timbers. It resembles the typical compression failure of a specimen in the testing machine.

Cross-grain, which may be spiral, diagonal, wavy, dip or curly, denotes the departure of the wood fibers from a direction parallel to the piece. Wavy, dip and curly grain are usually due to knots. Cross-grain may also result from diagonal sawing.

Knots are a structural defect, not that they are composed of material weaker than the normal wood, but because of a combination of several factors, chiefly cross-grain and checking.

Decay is the disintegration of wood substance due to fungi, which break down the cell walls. Oftentimes the visible decay does not indicate the extent to which the strength is affected. For this reason and because certain types of decay continue after timber is placed in service, it is advisable to limit decay more rigidly than tests might seem to justify.

The above defects must be controlled in specifications which establish definite strength grades. Other defects are for the most part indicative of the ones mentioned above. For example, *bark pockets* and *gum spots* and *streaks* indicate concealed knots; *pitch* indicates shakes and erratic strength; *insect holes* indicate decay; and *warping* indicates cross-grain.

Wane is bark, or lack of wood, on the edge or corner of a piece.

The fundamental strength properties of timber are: tension, compression and shear. Tension is affected most seriously by cross-grain, knots, cross breaks and " compression failure." The effect of cross-grain is large because of the great difference in the tension strength of wood with and across the grain, being about 25 to 1 for small, clear, green specimens and as high as 45 to 1 for air-dry specimens. The amount of weakening due to cross-grain depends on the angle the grain makes with the axis of the piece. A slight departure from straight grain is a source of weakness, but it is not pronounced until a slope of 1 in 40 is reached. A slope of 1 in 20 in a beam reduces the strength about one-eighth; 1 in 15, about one-fourth; 1 in 11, about three-eighths; and 1 in 8, about one-half.

The weakening effect of knots on tensile properties is mostly due to the cross-grain and checking introduced. The effect of the knot is about the same as that resulting from the removal of an equal amount of wood, and, in addition, the influence of the checking and irregular grain surrounding the knot must be taken into consideration.

Checks and shakes have no effect on tensile properties when the load is applied in line with the grain. However, with tension across the grain the strength is lost. Cross breaks and " compression failure" reduce the strength when the load is parallel to the grain.

In compression the effect of cross-grain and knots is much less than in tension. The effect of checks and shakes is to cause an unequal distribution of stresses. The compressive strength of wood parallel to the grain is from six to ten times that perpendicular to the grain. This is much less than the ratio given above for tension, hence cross-grain does not weaken compression members as much as it does tension members. In a post or column an angle of 1 in 15 reduces the strength about one-eighth; 1 in 11, about one-fourth; 1 in 8, about three-eighths; and 1 in 6, about one-half.

Checks in compression parallel to the grain tend to close up so that stress is set up over the entire cross-section. However, failures usually occur at somewhat lower average loads than when checks are not present.

The weakening effect of a knot in compression is due chiefly to the cross-grain that accompanies it. However, the weakening effect is less than for tension and checking around and in the knot is less injurious.

Reduction in shearing strength is due chiefly to the presence of shakes and checks; knots and cross-grain exert but little influence, and this influence may be for good as they break up the continuity of checks. Checks and shakes directly reduce the area acting in resistance to shear. Moreover, shakes are usually accompanied by a general weakness in bond between the annual-growth rings.

ART. 85. GRADE REQUIREMENTS

Grade requirements for structural timbers are too voluminous to be given in full in a text of this size and therefore only an outline of the character of the requirements will be presented. For complete specifications see the Manual of the American Railway Engineering Association or the Standards of the American Society for Testing Materials.

Shake is measured on the ends of a piece and the size is taken as the shortest distance between lines enclosing the shake and parallel to the wide face of the piece. Checks and splits are limited as provided for shakes. Shakes and checks must not exceed approximately the following fractions of the width of the end for green and seasoned conditions, respectively: dense-select and select joist and plank, one-fourth and one-third; dense-common and common joist and plank, four-tenths and four-ninths; dense-select and select beams and stringers, one-fourth and one-third; dense-common and common beams and stringers, four-tenths and four-ninths; dense-select and select posts and timbers, four-tenths and one-half; and dense-common and common posts and timbers, one-half and six-tenths.

The slope of grain in the center half of the length of the piece is limited as follows: dense-select and select joist and plank, 1 in 12; dense-common and common joist and plank, 1 in 10; dense-select and select beams and stringers, 1 in 15; dense-common and common beams and stringers, 1 in 10; dense-select and select posts and timbers, 1 in 10; and dense-common and common posts and timbers, 1 in 8.

Wane may or may not be permitted. If permitted it must not exceed one-eighth the width of any face for dense-select and select grades, nor one-fourth the width of any face for dense-common and common grades.

Heartwood and sapwood have been found by the Forest Products Laboratory to be of equal strength and no requirement for heartwood need be made when strength alone is the governing factor. Heartwood requirements, when durability of untreated material under exposure is a factor, as in bridges, trestles, docks and piers, damp buildings, etc., may be specified in any grade according to exposure and use. Where preservative treatment is to be applied, there should be no restriction as to sapwood, as sapwood is easier to treat than heartwood.

Among the heartwood requirements sometimes set up are the following: Joists and planks shall have not less than 85 percent of heart on the two faces as measured across the faces anywhere in the length of the piece; beams and stringers shall have not less than 85 percent heart on each of the four faces measured across the faces anywhere in the length of the piece; posts and timbers, one of the following classes—(1) not less than 85 percent heart on each of the four faces, (2) all heart on one narrow face and not less than 85 percent heart on the other narrow face and on the two sides, and (3) all heart on one narrow face and 75 percent heart on the other narrow face and on the two sides. In all cases the measurements are across the faces or sides anywhere in the length of the piece.

The select grade of Douglas fir is based on closeness of grain—rate of growth—of the wood, whereas the dense-select of Douglas fir and southern pine is based on density. Select Douglas fir shall be of close grain, averaging on either one end or the other not less than six nor more than twenty annular rings measured over a 3-in. portion of a radial line. Pieces averaging from five to six annular rings per inch shall be accepted as the equivalent of close grain if they have one-third or more summerwood. Dense-select southern pine and Douglas fir shall be dense, averaging on either one end or the other not less than six annular rings per inch and, in addition, one-third or more summerwood measured over a 3-in. portion of a radial line. The contrast in color between summerwood and springwood shall be distinct. Coarse-grained material excluded by this rule shall be accepted as dense if averaging one-half or more summerwood.

In boxed-heart pieces of southern pine the measurement shall be made over the third, fourth and fifth inches from the pith along a radial line. Where timbers do not contain the pith and it is impossible to locate it with any degree of accuracy, the inspection shall be made over 3 in. on an approximate radial line beginning at the edge nearest the pith in timbers over 3 in. in thickness and on the second inch nearest to the pith in timbers 3 in. or less in thickness. In material containing the pith but not a 5-in. radial line, which is less than 2 by 8 in. in section or less than 8 in. in width, that does not show over 16 sq. in. on the cross-section, the inspection shall apply to the second inch from the pith. In larger material that does not show a 5-in. radial line, the inspection shall apply to the 3 in. farthest from the pith.

In boxed-heart pieces of Douglas fir the radial line shall run from the pith to the corner farthest from the pith. When the least dimension is 6 in. or less, the 3-in. portion of the line shall begin at a distance of 1 in. from the pith. When the least dimension is more than 6 in., the 3-in. portion of the line shall begin at a distance from the pith equal to 2 in. less than one-half the least dimension of the piece. In sidecut pieces of Douglas fir the radial line shall be at a right angle to the annular rings and the center of the 3-in. portion of the line shall be at the center of the end of the piece. If a 3-in. portion of the radial line cannot be obtained, the measurement shall be over as much of the 3-in. portion as is available.

Where the radial line specified is not representative, it shall be shifted sufficiently to present a fair average, but the distance from the pith to the beginning of the 3-in. portion of the line in boxed-heart pieces shall not be changed. In case of disagreement in the measurement of close grain or density, two radial lines shall be chosen, and the number of rings or number of rings and percentage of summerwood shall be the average determined on these lines.

In grading timber with reference to knots, their size and position are of chief importance. Cluster knots and knots in groups are not permitted. In joists and planks the size of knots in inches on wide faces is limited as follows:

Width of Face, Inches	On or Near Edge, Middle Third of Length		Center Line of Face	
	Dense-Select and Select	Dense-Common and Common	Dense-Select and Select	Dense-Common and Common
4	$\frac{3}{4}$	1	$1\frac{1}{4}$	$1\frac{3}{4}$
6	1	$1\frac{1}{2}$	2	$2\frac{1}{2}$
8	$1\frac{3}{8}$	2	$2\frac{5}{8}$	$3\frac{3}{8}$
10	$1\frac{3}{4}$	$2\frac{1}{2}$	$3\frac{1}{4}$	$4\frac{1}{4}$
12	$2\frac{1}{8}$	3	4	$5\frac{1}{8}$
14	$2\frac{3}{8}$	$3\frac{1}{4}$	$4\frac{1}{4}$	$5\frac{5}{8}$
16	$2\frac{1}{2}$	$3\frac{3}{8}$	$4\frac{5}{8}$	6

On narrow faces of boxed-heart pieces of joists and planks the size of knots in the middle third of the length are limited as follows, the first value being for dense-select and select grades and the second value for dense common and common grades: 2-in. thickness, $\frac{5}{8}$ and $\frac{7}{8}$ in.; 3-in. thickness, 1 and $1\frac{1}{4}$ in.; and 4-in. thickness, $1\frac{1}{4}$ and $1\frac{3}{4}$ in.

On the wide faces of joists and planks the measurement of the knot is taken as the average of its maximum and minimum diameters, and on the narrow face as its width between lines parallel to the edges of the piece. The size of knots of the narrow faces and edges of wide faces may increase proportionately from the size allowed in the middle third to twice that size at the ends of the pieces; on the wide faces the size may increase proportionately from the size allowed at the edge to that allowed at the center line. For dense-select and select joist and plank the sum of the diameters of all knots within the center half on any face shall not exceed one and one-half times the width of face on which they occur, and for dense-common and common joist and plank the sum of these diameters shall not exceed two times the width of face on which they occur.

In beams and stringers the sizes of knots in inches are limited as follows:

Width of Face, Inches	Narrow or Horizontal Face, Middle Third of Length		Center Line of Wide or Vertical Face	
	Dense-Select and Select	Dense-Common and Common	Dense-Select and Select	Dense-Common and Common
5	1¼	2	1¼	2
6	1½	2⅜	1½	2⅜
8	1¾	2¾	2	3⅛
10	2	3⅛	2½	4
12	2⅛	3⅜	3	4¾
14	2¼	3⅝	3¼	5⅛
16	2⅜	3⅞	3⅜	5½
18	....	....	3⅝	5⅞
20	....	....	3⅞	6⅛
22	....	....	4	6½
24	....	....	4¼	6¾

On the narrow or horizontal faces of beams and stringers the size of the knot is taken as its width between lines parallel to the edges of the timber; on the wide or vertical faces the smallest diameter of the knot is taken as its size. Knots on the edges of wide or vertical faces are limited to the same size as on the adjacent narrow or horizontal faces, except that the size is measured on the least diameter of the knot instead of on its width between lines parallel to the edges of the timber. The size of knots on the narrow or horizontal faces and edges of wide or vertical faces may increase proportionately from the size allowed in the middle third to twice that size at the ends of the piece. The size of knots on the wide or vertical faces may increase proportionately from the size allowed at the edge to that allowed at the center line. For dense select and select beams and stringers the sum of the diameters of all knots within the center half of the length on any face shall not exceed the width of face on which they occur and for dense-common and common beams and stringers the sum of the diameters shall not exceed one and one-half times the width of face on which they occur.

In posts and timbers the sizes of knots in inches, taken as the average of the maximum and minimum diameters, are limited as follows:

Width of Face, Inches....	6	8	10	12	14	16	18	20	22	24
Size of Knot, Dense-Select and Select............	1½	2	2½	3	3¼	3⅜	3⅝	3⅞	4	4¼
Size of Knot, Common...	2⅜	3⅛	4	4¾	5⅛	5½	5⅞	6⅛	6½	6¾

The sum of the diameters of all knots in any 6 in. of length of a post or timber shall not exceed twice the size of the maximum knot allowable.

ART. 86. PROPERTIES OF CLEAR TIMBER

By testing small clear specimens of wood the influence of defects may be eliminated from calculations to determine the relation between strength and density, moisture, locality of growth, etc. In Bulletin 556 of the Forest Service, entitled Mechanical Properties of Wood Grown in the United States, there are given the results of tests on about 130,000 clear specimens, 2 by 2 in. in cross-section and up to 30 in. in length.

The test results are given in two tables, one for green timber and one for air-dry timber. Data are given for eighty-eight species of hardwoods and thirty-eight species of conifers, the information presented covering locality where grown, moisture content, unit weight, specific gravity, shrinkage, static bending, impact bending, compressive strength, shearing strength, tensile strength and hardness.

The locality where grown is stated because in some cases this has an influence on the strength of timber, although usually as much difference exists between stands in the same locality as between stands in different states. Strength tests on Douglas fir show a considerable difference between the Rocky Mountain and Pacific Coast types.

The moisture content is expressed in percent of the oven-dry weight of the wood. Specific gravity is given for oven-dry wood based on volume when green, volume when oven-dry and volume when air-dry. Unit weights are given for green and for air-dry conditions. Shrinkage is given from green to oven-dry conditions, volumetrically, radially and tangentially.

Under static bending, data are given on fiber stress at elastic limit, modulus of rupture, modulus of elasticity, work in bending to elastic limit, and work in bending to maximum load. In the impact bending tests a weight was dropped on a beam, the height of fall increasing from zero to a maximum, values being given for the fiber stress at elastic limit, work in bending to elastic limit and height of drop causing complete failure.

The unit strength in compression parallel to the grain is given, both for elastic limit and ultimate load conditions; and for compression perpendicular to the grain values are given for elastic limit. Shearing strength parallel to the grain and tension perpendicular to the grain are given for ultimate load conditions. Hardness test results are given for loads on the ends and on the sides of the fibers.

Tables 86*a* and 86*b*, taken from the above mentioned bulletin, give test results of some of the more widely used woods. There is a large variation in the moisture content of the different species given in Table 86*a*. However, all tests were made at approximately the moisture content of the living tree—although the normal content varies somewhat in different parts of the tree—and are well above the limit below which differences in moisture content produce differences in strength. Table 86*a* is the more reliable index of the comparative qualities of the several species because it is based on a larger number of tests and on tests that are not influenced by variations in moisture content.

Many of the properties of air-dry wood are subject to rapid change with change in moisture content. The strength properties will vary from 4 to 6 percent for each percent of moisture greater or less than that given in Table 86*b*. For this reason in comparing species in Table 86*b* it is necessary to make allowances for the differences shown in the column of moisture content.

TABLE 86*a*.—TESTS OF SMALL CLEAR

[Test specimens are 2 by 2 in. in section. Bending specimens

Name	Locality	Number of trees	Number of rings per inch	Summerwood (percent)	Moisture content (percent)	Specific Gravity, Oven dry, Based On—		Weight per cubic foot (green)	Shrinkage from Green to Oven-dry		
						Volume, green	Volume oven-dry		In volume (percent of dimensions green)	Radial (percent of dimensions green)	Tangential (percent of dimensions green)
Cedar, Port Orford	Oregon	5	24	25	52	.41	.47	39	10.7	5.2	8.1
Cedar, western red	Washington, Montana	10	20	36	39	.31	.34	27	8.1	2.5	5.1
Cedar, white	Wisconsin	5	23	36	55	.29	.32	28	7.0	2.1	4.9
Cypress, bald	Louisiana, Missouri	10	16	31	87	.41	.47	48	10.7	3.8	6.0
Douglas fir	Washington, Oregon	18	13	35	36	.45	.52	38	12.6	5.0	7.9
Douglas fir	Montana, Wyoming	10	22	27	38	.40	.44	34	10.6	3.6	6.2
Fir, balsam	Wisconsin	5	12	26	117	.34	.41	45	10.8	2.8	6.6
Fir, white	California	5	10	30	156	.35	.44	56	10.2	3.4	7.0
Hemlock (eastern)	Tennessee, Wisconsin	10	20	34	105	0.38	0.44	48	10.4	3.0	6.4
Hemlock (western)	Washington	5	10	27	71	.38	.43	41	11.6	4.5	7.9
Larch, western	Montana, Washington	13	32	37	58	.48	.59	48	13.2	4.2	8.1
Oak, red	Arkansas, Louisiana, Indiana, Tennessee	21	11	62	84	.56	.65	64	14.2	3.9	8.3
Oak, white	Arkansas, Louisiana, Indiana	20	17	60	68	.60	.71	62	15.8	5.3	9.0
Pine, loblolly	Florida, North Carolina, South Carolina	15	8	42	70	.50	.59	54	12.6	5.5	7.5
Pine, longleaf	Florida, Louisiana, Mississippi	34	18	39	47	.55	.64	50	12.3	5.3	7.5
Pine, Norway	Wisconsin	5	22	41	54	.44	.51	42	11.5	4.6	7.2
Pine, shortleaf	Arkansas, Louisiana	12	12	40	64	.50	.58	50	12.6	5.1	8.2
Pine, sugar	California	5	12	34	123	.36	.39	50	8.4	2.9	5.6
Spruce, Engelmann	Colorado	10	14	14	100	.31	.35	38	10.4	3.4	6.6
Spruce, Sitka	Washington	5	9	24	53	.34	.37	33	11.2	4.5	7.4
Spruce, white	New Hampshire, Wisconsin	7	14	27	46	.36	.43	33	14.8	3.7	7.3
Tamarack	Wisconsin	5	20	38	52	.49	.56	47	13.6	3.7	7.4

Pieces of Green Timber

are cut 30 in. long; others are shorter, depending on kind of test.]

Static Bending					Impact Bending			Compression Parallel to Grain					Hardness, Load Required to Embed a 0.444-in. Ball to One-half Its Diameter	
			Work in Bending											
Fiber stress at elastic limit (pounds per square inch)	Modulus of rupture (pounds per square inch)	Modulus of elasticity (1,000 pounds per square inch)	To elastic limit (inch-pounds per cubic inch)	To maximum load (inch-pounds per cubic inch)	Fiber stress at elastic limit (pounds per square inch)	Work in bending to elastic limit (inch-pounds per cubic inch)	Height of drop causing complete failure, 50-pound hammer (inches)	Fiber stress at elastic limit (pounds per square inch)	Maximum crushing strength (pounds per square inch)	Compression perpendicular to grain—fiber stress elastic limit (pounds per square inch)	Shearing strength parallel to grain (pounds per square inch)	Tension perpendicular to grain (pounds per square inch)	End (pounds)	Side (pounds)
3900	6800	1500	59	7.8	9,300	2.7	25	2970	3230	380	880	240	560	480
3300	5200	950	.64	5.0	7,100	2.4	17	2500	2840	310	720	210	430	260
2600	4200	640	.60	5.7	5,300	2.0	15	1420	1990	290	620	240	320	230
4000	6800	1190	.86	6.4	8,000	2.6	24	3100	3490	470	820	280	470	380
5000	7800	1580	.86	6.7	9,400	2.9	25	3400	3940	530	910	200	510	470
3600	6400	1180	.65	6.8	9,100	3.0	20	2520	3000	450	880	350	450	400
3000	4900	960	.52	4.7	6,900	2.3	16	2220	2400	210	610	180	290	290
3900	6000	1130	.77	5.2	7,200	2.2	18	2610	2800	440	730	260	380	330
4200	6700	1120	0.88	6.8	7,900	2.8	20	2710	3270	500	880	260	510	410
3400	6100	1190	.58	6.0	7,800	2.4	20	2290	2890	350	810	260	540	430
4600	7500	1350	1.01	7.1	9,400	3.7	24	3250	3800	560	920	230	470	450
3700	7700	1290	.65	11.5	10,400	3.9	41	2330	3200	730	1120	740	1020	950
4700	8300	1250	1.08	11.5	10,700	4.2	42	2990	3560	830	1250	770	1120	1060
4400	7500	1380	.81	8.0	9,500	3.1	32	2870	3580	550	900	280	400	450
5400	8700	1630	1.00	8.0	10,800	3.5	34	3840	4390	600	1070	290	550	590
3700	6400	1380	.59	5.8	7,500	2.2	28	2470	3080	360	780	190	360	340
4500	8000	1450	.79	8.7	11,200	4.0	39	3650	3810	480	890	330	490	560
3300	5300	970	.66	5.0	6,700	2.3	17	2340	2600	350	710	270	330	320
2500	4200	830	.43	4.9	5,800	1.9	14	1740	1980	290	590	....	250	240
3000	5500	1180	.44	6.4	7,900	2.5	29	2280	2600	330	780	230	430	370
3300	5400	980	.66	5.7	6,800	2.0	20	2280	2380	270	670	200	300	280
4200	7200	1240	.84	7.2	7,800	2.7	28	3010	3480	480	860	260	400	380

TABLE 86*b*.—TESTS OF SMALL CLEAR

[Test specimens are 2 by 2 in. in section. Bending specimens

Common and Botanical Name	Locality Where Grown	Number of rings per inch	Summerwood (percent)	Moisture content (percent)	Specific gravity, oven-dry, based on volume when air dry	Weight per cubic foot (air dry) (pounds)	Static Bending		
							Fiber stress at elastic limit (pounds per square inch)	Modulus of rupture (pounds per square inch)	Modulus of elasticity (1,000 pounds per square inch)
Cedar, Port Orford	Oregon	24	25	9.0	.45	31	8,900	14,500	2040
Cedar, western red	Washington, Montana	20	36	7.4	0.34	22	6,100	8,080	1250
Cedar, white	Wisconsin	23	36	11.2	.31	22	5,100	6,700	810
Cypress, bald	Louisiana, Missouri	16	31	9.0	.44	30	8,700	11,300	1540
Douglas fir	Montana, Wyoming	22	27	9.4	.44	30	6,900	10,300	1460
Douglas fir	Washington, Oregon	13	35	6.2	.51	34	10,600	14,000	2210
Fir, balsam	Wisconsin	12	26	4.8	.38	25	7,300	9,900	1440
Fir, white	California	10	30	9.6	.38	26	7,000	9,800	1490
Hemlock (eastern)	Tennessee, Wisconsin	20	34	8.6	.42	28	7,200	9,700	1300
Hemlock (western)	Washington	10	27	5.4	.42	28	8,000	10,800	1520
Larch, western	Montana, Washington	36	37	8.3	.54	36	10,100	13,500	1830
Oak, red	Arkansas, Louisiana, Indiana, Tennessee	11	62	10.9	.63	44	8,600	14,200	1870
Oak, white	Arkansas, Louisiana, Indiana	17	60	11.5	.69	48	8,300	15,200	1780
Pine loblolly (Pinus tæda)	Florida	9	42	6.5	.57	38	11,700	15,600	2130
Pine, longleaf	Florida, Louisiana, Mississippi	18	38	9.2	.66	42	11,800	16,700	2200
Pine, Norway	Wisconsin	22	41	12.5	.48	34	9,200	12,300	1790
Pine, shortleaf	Arkansas, Louisiana	10	40	11.0	.54	38	9,200	13,900	1970
Pine, sugar	California	12	34	11.4	.37	26	6,400	8,600	1210
Spruce, Engelmann	Colorado	14	14	14.8	.32	24	4,500	6,800	1030
Spruce, Sitka	Washington	9	24	8.9	.38	26	7,200	11,200	1610
Spruce, white	New Hampshire, Wisconsin	14	27	9.6	.40	28	5,900	9,200	1390
Tamarack	Wisconsin	20	38	11.0	.53	37	8,400	12,000	1680

Pieces of Air-Dry Timber
are cut 30 in. long; others are shorter, depending on kind of test.]

Static Bending Continued		Impact Bending			Compression Parallel to Grain					Hardness, Load Required to Embed a 0.444-in. Ball to One-half Its Diameter	
Work in Bending—											
To elastic limit (inch-pounds per cubic inch)	To maximum load (inch-pounds per cubic inch)	Fiber stress at elastic limit (pounds per square inch)	Work to elastic limit (inch-pounds per cubic inch)	Height of drop causing complete failure, 50-pound hammer (inches)	Fiber stress at elastic limit (pounds per square inch)	Maximum crushing strength (pounds per square inch)	Compression perpendicular to grain—fiber stress at elastic limit (pounds per square inch)	Shearing strength parallel to the grain (pounds per square inch)	Tension perpendicular to grain (pounds per square inch)	End (pounds)	Side (pounds)
2.25	....	17,600	7.2	39	7700	7,750	1020	1500	630	950	700
1.68	6.4	9,600	3.4	16	5390	6,320	700	920	180	750	380
1.84	4.7	7,200	2.8	12	2840	4,140	390	900	240	470	340
3.03	7.3	11,200	4.7	26	5970	7,690	910	1080	280	800	550
1.83	6.5	13,300	5.6	28	5320	7,090	950	1080	320	750	660
2.94	8.4	14,300	5.4	33	9260	10,680	1220	1270	330	920	810
2.11	5.4	8,500	2.9	23	......	6,640	530	790	180	740	500
1.86	5.5	8,400	3.2	14	4180	6,150	720	1060	260	780	460
2.32	5.8	12,500	6.0	24	5240	7,060	1060	1160	190	860	490
2.48	6.1	13,000	6.0	26	7780	7,910	830	1170	170	1020	620
3.06	8.2	17,000	8.6	34	8420	9,640	1280	1530	340	1380	870
2.32	13.3	18,500	9.1	39	4610	7,370	1210	1760	780	1500	1310
2.31	14.8	17,200	7.6	38	4350	7,610	1340	2090	800	1540	1370
3.70	9.0	14,800	5.9	26	7980	11,300	1600	1720	370	1030	840
3.58	11.3	16,400	6.6	32	9250	10,880	1670	1640	420	1140	1020
2.68	9.9	15,100	6.3	25	5660	7,080	830	1260	470	700	600
2.46	10.1	16,600	6.5	36	7080	8,660	1310	1390	410	940	880
1.79	5.0	10,100	4.3	17	4740	5,190	640	1080	350	650	460
1.09	5.4	8,300	3.2	15	2980	3,810	520	910	.....	390	290
1.78	10.4	13,900	5.2	25	......	5,770	1010	1210	.....	780	530
1.40	8.2	9,700	3.5	20	4620	6,020	590	970	350	760	560
2.35	7.1	13,000	5.7	23	5150	7,590	1080	1370	410	720	640

The weights given for air-dry material are for wood with 12 percent moisture. Large timbers ordinarily have more than this amount, averaging from 10 to 15 percent heavier than the listed weights. As a rule, conifers dry much more rapidly than hardwoods. Small-size timbers will, in a heated room, dry to 6 percent moisture, whereas the same material, under shelter out of doors, seldom has less than 12 percent moisture. Moisture contents varying from 15 to 25 percent are common in air-dry material under shelter out of doors.

A comparison of the accompanying tables shows that the strength properties of air-dry specimens are from one-third to almost three times those of green specimens. The modulus of rupture in static bending of air-dry Douglas fir is 32 percent greater than that of green Douglas fir; and for longleaf pine it is 89 percent. The increase in crushing strength parallel to the grain is 80 percent for Douglas fir and 149 percent for longleaf pine; and for compression perpendicular to the grain at elastic limit, these values are 79 percent and 178 percent, respectively, for Douglas fir and longleaf pine.

ART. 87. TESTS OF TIMBERS OF COMMERCIAL SIZE

The subject of this article is so extensive that it cannot be adequately treated in a book of this size. Some data will be given, but for further information the reader is referred to the bibliography at the end of this chapter.

Table 87*a*, arranged from material taken from Bulletin 108 of the Forest Service, entitled Tests of Structural Timbers, gives results of tests on some green and air-dry timbers of commercial size. Each value is the average of a number of tests, the sizes varying from 2 by 8 in. to 12 by 12 in. and 10 by 16 in. For the bending tests, in the majority of cases, the span was 12 ft.

In practically all cases the air-dry specimens are stronger than the green ones, but the difference is not so great as in the small clear tests described in the preceding article. For example, the modulus of rupture of air-dry Douglas fir is 6 percent greater than for green material; but for longleaf pine the green specimens are slightly stronger than the dry ones. For Douglas fir in compression parallel to the grain the air-dry specimens are 22 percent stronger than the green ones; for compression perpendicular to the grain at elastic limit the excess is 12 percent. For longleaf pine the compressive strengths in both directions are about the same for air-dry and green specimens. The air-dry specimens, based on an average of all the species listed in Table 87*a*, have a modulus of rupture 14 percent, and a compressive strength parallel to the fiber 40 percent, greater than the green specimens.

The reason for the smaller differences between air-dry and green strengths for commercial-size material as given in Table 87*a* is the development of defects in drying, as well as the larger moisture content; compare with the test results of Tables 86*a* and 86*b*.

The small clear specimens of a given species are stronger than the commercial sizes for practically all methods of load application. For example, in the modulus of rupture tests, the green small clear pieces are stronger than the commercial sizes by 30 percent for Douglas fir, by 41 percent for longleaf pine, and by 43 percent for the average (excluding redwood) of all species given in Table 87*a*. For compression parallel to the grain the figures are 13, −9, and 6 percent, respectively.

Comparing the air-dry tests, for modulus of rupture the small clear pieces are stronger than the commercial sizes by 62 percent for Douglas fir, by 192 percent for longleaf pine, and by 116 percent for the average of all species. For compression parallel to the grain, the results are 67, 126 and 77 percent, respectively. The reasons for the greater difference of strength in the air-dry specimens are: (1) the greater development of checks, and (2) the larger amount of moisture in the commercial-size timbers.

The bulletin mentioned at the beginning of this article contains valuable data on the variation of individual tests, which indicates, as noted in Art. 88, that in a material as non-homogeneous as wood an average result is only of general significance.

A study of these tests show that, of green timbers, from 18 to 43 percent failed first in tension, from 50 to 76 percent failed in compression, and from 1 to 17 percent failed in horizontal shear, except longleaf pine, where the percentage of shear failures was 58. Of air-seasoned timbers from 11 to 83 percent failed first in tension, from 12 to 76 percent failed first in compression, and from 5 to 56 percent failed in horizontal shear.

Of the tension failures, 38 percent were due to knots, 52 percent to irregular grain, 2 percent to pitch pockets, and 8 percent to no apparent cause. Compression failures in a large proportion of

TABLE 87*a*.—TESTS OF TIMBERS OF COMMERCIAL SIZE

	Percent of Moisture Based on Bending Test Specimens	Weight per Cu. Ft. Oven-dry	Rings per In.	Bending				Compression Parallel to Grain			Compression Perpendicular to Grain
				Fiber Stress at Elastic Limit, per Sq. In.	Modulus of Rupture per Sq. In.	Modulus of Elasticity, per Sq. In.	Horizontal Shear, per Sq. In.*	Crushing Strength at Elastic Limit, per Sq. In.	Crushing Strength at Maximum Load, per Sq. In.	Modulus of Elasticity, per Sq. In.	Crushing Strength at Elastic Limit, per Sq. In.
		Lbs.		Lbs.	Lbs.	1000 Lb.	Lbs.	Lbs.	Lbs.	1000 Lb.	Lbs.
Douglas fir:											
Green	29	28	11.0	3968	5983	1517	166	2770	3495	1414	570
Air-seasoned	21	28	13.1	4563	6372	1549	221	3271	4258	1038	639
Hemlock, western:											
Green	43	27	15.6	3516	5296	1445	288	2905	3355	1617	434
Air-seasoned	17	28	17.7	4398	6420	1737	307	4840	5814	2140	473
Larch, western:											
Green	51	28	24.3	3324	4948	1301	288	2675	3510	1575	456
Air-seasoned	17	29	22.7	3503	5856	1487	340	....	5746	....	597
Pine, loblolly:											
Green	59	31	5.9	3040	5084	1387	325	2050	2940	548	500
Air-seasoned	21	32	6.3	3517	6118	1487	434	3011	4292	1206	655
Pine, longleaf:											
Green	29	35	13.8	3734	6140	1463	353	3480	4800	....	568
Air-seasoned	21	39	15.4	3691	5749	1705	272	3480	4800	....	572
Pine, Norway:											
Green	48	25	13.7	2492	3864	1133	232	2065	2555	1002	
Air-seasoned	15	27	10.0	4069	6054	1418	278	3047	4228	1367	
Pine, shortleaf:											
Green	48	30	12.1	3237	5548	1473	332	2460	3435	1548	351
Air-seasoned	15	31	12.4	4675	6573	1726	364	4070	6030	1951	790
Redwood:											
Green	76	22	18.8	3760	4472	1042	302	3194	3882	1240	434
Air-seasoned	16	22	20.1	3442	3891	890	....	....	4276	....	525
Tamarack:											
Green	50	30	14.0	2813	4556	1220	261	2400	3230	1373	
Air-seasoned	19	31	12.7	3730	5498	1341	299	3349	4320	1351	

* Only those pieces failing first in horizontal shear are included in this column.

the tests were apparently not influenced by any defect. Most of the horizontal shear failures in the air-dry material were due to checks and this is probably also true for the green material although the checks were not visible.

ART. 88. FACTORS OF SAFETY

The unit stress to be adopted for design depends on the factor of safety that should be applied to the natural strength of the timber. This factor of safety will depend upon: (1) the variability of strength of the clear wood, (2) the effect of moisture on strength, (3) the duration of stress, and (4) the presence of defects.

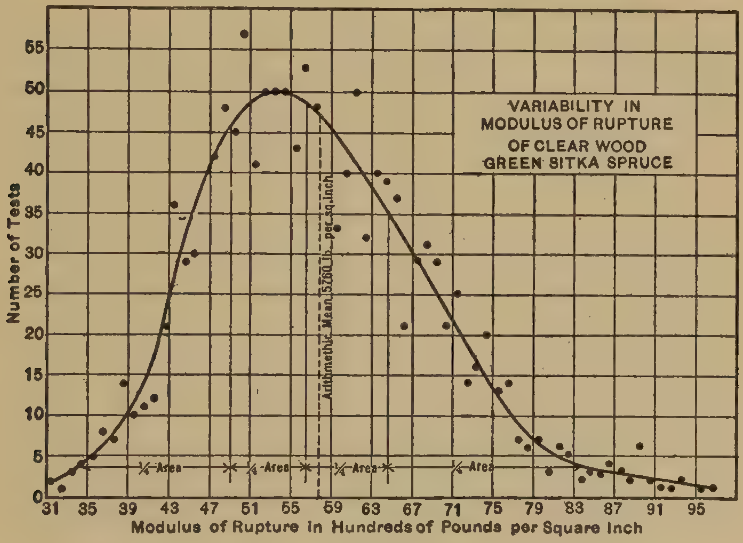


FIG. 88*a*.

Even in clear wood different specimens of the same kind of timber will vary widely in strength. This variation is well illustrated by Fig. 88*a*,[1] which represents the variability in modulus of rupture of Sitka spruce in a green condition. This curve is characteristic of all variability curves for wood. The curve is not symmetrical, the greater number of tests giving values below the average and the greater range of variation lying above the average, whereas the most probable value, corresponding to the highest point on the curve, is slightly below the average. In assigning working stresses, primary consideration is given to the one-fourth of the pieces lowest in strength.

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[1] Proceedings American Society of Civil Engineers, Sept., 1926.

The weakening effect of moisture on timber has been discussed in the preceding articles. Figure 88*b*,[1] shows the results of some tests made on some small-size pieces of high-grade spruce to determine the effect of moisture. Air-seasoned spruce carries about 10 percent moisture, which indicates that the bending strength of air-seasoned spruce is about twice that of green spruce. Such tests as these show the necessity for using lower design stresses where timber is exposed to the weather.

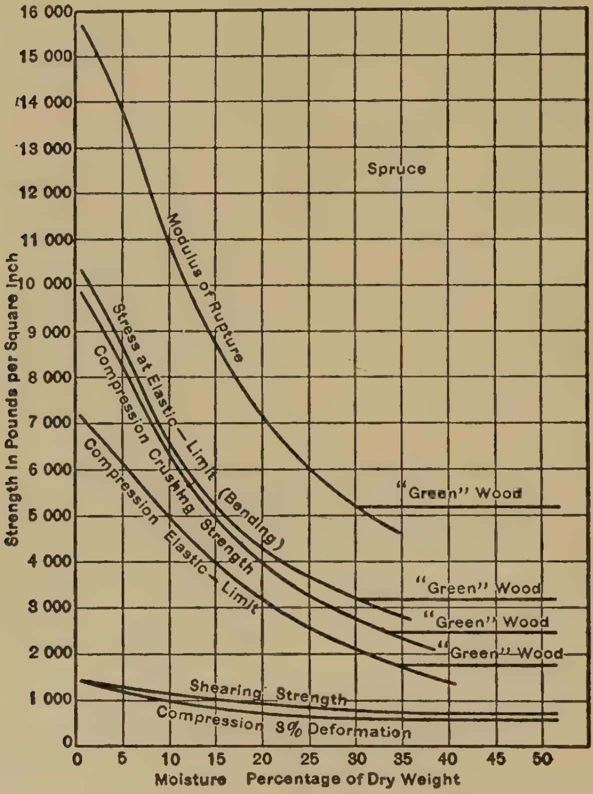

FIG. 88*b*.—Effect of Moisture on Strength of Spruce.

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[1] Proceedings American Society of Civil Engineers, Sept., 1926.

Tests show that a load that can safely be carried for a short time may ultimately cause failure on a piece of timber, this load sometimes being not over half the breaking strength of the wood. As most tests are carried out under a short-time loading condition this effect must be considered in fixing a proper factor of safety.

As explained in the preceding articles full-size timbers are not as strong as small clear pieces owing to the presence of defects. A strict adherence to grading rules will assure strengths approaching those of small clear pieces, but consideration must be given this fact when a factor of safety is chosen based on test results of small clear pieces.

The following example illustrates how a safe designing unit stress may be developed from a knowledge of the strength of a small clear piece of a certain species of timber: The average modulus of rupture for small clear spruce is 5500 lb. per sq. in., when tested green at standard speed. To get a safe working stress for a select grade, we will reduce this by one-fourth to take care of the variability of the clear wood, and this value will be again reduced by one-fourth to take care of maximum defects permitted by the grading rules. The latter value will be reduced one-half to take care of the effect of long-time loading, the resulting value being $5500 \times \frac{3}{4} \times \frac{3}{4} \times \frac{1}{2} = 1550$ lb. per sq. in. Hence the recommended stresses (Art. 89) of 800 and 710 lb. per sq. in. for timber occasionally wet but quickly dried and continuously damp or wet, respectively, give factors of safety of 1.9 and 2.2, respectively.

ART. 89. WORKING UNIT STRESSES

The stresses given in Table 89*a* are those recommended by the Forest Products Laboratory, U. S. Forest Service, and adopted as standard in 1927 by the American Railway Engineering Association and by the American Society for Testing Materials. These stresses are for material complying with the structural grades as adopted by these associations as outlined in Art. 85.

The defects permitted in the common grades provide material having not less than 60 percent of the strength of green clear wood and in the select grades not less than 75 percent. As noted before, timber constantly yields under long-continued loading, acquiring a permanent set. This set, with a fully loaded beam, is about equal to the deflection using the modulus of elasticity as

Table 89*a*.—Working Stresses for Joists and Planks and Beams and Stringers

	Extreme Fiber in Bending					Compression Perpendicular to Grain			Maximum Horizontal Shear	Modulus of Elasticity
		4 In. and Thicker	5 In. and Thicker	4 In. and Thicker	5 In. and Thicker					
Select Grade	a	b	b	c	c	a	b	c	a-b-c	a-b-c
Cedar; western red	900	710	800	670	750	200	150	125	80	1,000,000
Northern and southern white	750	580	....	530	....	175	140	100	70	800,000
Port Orford	1100	890	1000	800	900	250	200	150	90	1,200,000
Alaska	1100	890	....	800	....	250	200	150	90	1,200,000
Cypress, southern	1300	980	....	800	....	350	250	225	100	1,200,000
Douglas fir:										
Coast region	1600	1240	1385	950	1065	345	240	215	90	1,600,000
Dense	1750	1370	1515	1050	1165	380	265	235	105	1,600,000
Rocky Mountain region	1100	800	900	620	700	275	225	200	85	1,200,000
Fir; balsam	900	670	....	530	....	150	125	100	70	1,000,000
Golden, noble, silver, white	1100	800	....	710	....	300	225	200	70	1,100,000
Hemlock; west coast	1300	980	1100	800	900	300	225	200	75	1,400,000
Eastern	1100	800	....	710	....	300	225	200	70	1,100,000
Larch; western	1200	980	1100	800	900	325	225	200	100	1,300,000
Pine; southern, dense	1750	1370	1515	1050	1165	380	265	235	128	1,600,000
California, Idaho and northern white, Pondosa and sugar	900	710	....	670	....	250	150	125	85	1,000,000
Norway	1100	890	....	710	....	300	175	150	85	1,200,000
Redwood	1200	....	1000	710	800	250	150	125	70	1,200,000
Spruce; red, white, Sitka	1100	800	900	710	800	250	150	125	85	1,200,000

Englemann	750	580	....	440	....	175	140	100	70	800,000
Tamarack, eastern	1200	980	....	800	....	300	225	200	95	1,300,000
Common Grade										
Cedar; western red	720	600	640	570	600	200	150	125	64	1,000,000
Northern and southern white	600	490	....	450	....	175	140	100	56	800,000
Port Orford	880	760	800	680	720	250	200	150	72	1,200,000
Alaska	880	760	....	680	....	250	200	150	72	1,200,000
Cypress; southern	1040	830	....	680	....	350	250	225	80	1,200,000
Douglas fir; coast region	1200	980	1040	750	800	325	225	200	72	1,600,000
Dense	1400	1145	1210	875	930	380	265	235	84	1,600,000
Rocky Mountain region	880	680	720	530	560	275	225	200	68	1,200,000
Fir; balsam	720	570	....	450	....	150	125	100	56	1,000,000
Golden, noble, silver, white	880	680	....	600	....	300	225	200	56	1,100,000
Hemlock; west coast	1040	830	880	680	720	300	225	200	60	1,400,000
Eastern	880	680	....	600	....	300	225	200	56	1,100,000
Larch; western	960	830	880	680	720	325	225	200	80	1,300,000
Pine; southern	1200	980	1040	750	800	325	225	200	88	1,600,000
Dense	1400	1145	1210	875	930	380	265	235	103	1,600,000
California, Idaho and northern white.										
Pondosa and sugar	720	600	....	570	....	250	150	125	68	1,000,000
Norway	880	760	....	600	....	300	175	150	68	1,200,000
Redwood	960	760	800	600	640	250	150	125	56	1,200,000
Spruce; red, white, Sitka	880	680	720	600	640	250	150	125	68	1,200,000
Englemann	600	490	....	370	....	175	140	100	56	800,000
Tamarack; eastern	960	830	....	680	....	300	225	200	76	1,300,000

Use values in Column (a) where timber is continuously dry.
Use values in Column (b) where timber is occasionally wet but quickly dried.
Use values in Column (c), where timber is more or less continuously damp or wet.

TABLE 89*b*.—UNIT STRESSES FOR STRUCTURAL TIMBER EXPRESSED IN POUNDS PER SQUARE INCH

Kind of Timber	Bending			Shearing				Compression						Ratio of Length of Stringer to Depth
	Extreme Fiber Stress		Modulus of Elasticity	Parallel to the Grain		Longitudinal Shear in Beams		Perpendicular to the Grain		Parallel to the Grain		For Columns under 15 Diams. Working Stress	Formulas for Working Stress in Long Columns over 15 Diameters	
	Average Ultimate	Working Stress	Average	Average Ultimate	Working Stress	Average Ultimate	Working Stress	Elastic Limit	Working Stress	Average Ultimate	Working Stress			
Douglas fir.......	6100	1200	1,510,000	690	170	270	110	630	310	3600	1200	900	$1200(1 - l/60d)$	10
Longleaf pine.....	6500	1300	1,610,000	720	180	300	120	520	260	3800	1300	980	$1300(1 - l/60d)$	10
Shortlead pine....	5600	1100	1,480,000	710	170	330	130	340	170	3400	1100	830	$1100(1 - l/60d)$	10
White pine.......	4400	900	1,130,000	400	100	180	70	290	150	3000	1000	750	$1000(1 - l/60d)$	10
Spruce...........	4800	1000	1,310,000	600	150	170	70	370	180	3200	1100	830	$1100(1 - l/60d)$	
Norway pine......	4200	800	1,190,000	590*	130	250	100	....	150	2600*	800	600	$800(1 - l/60d)$	
Tamarack........	4600	900	1,220,000	670	170	260	100	....	220	3200*	1000	750	$1000(1 - l/60d)$	
Western hemlock..	5800	1100	1,480,000	630	160	270*	100	440	220	3500	1200	900	$1200(1 - l/60d)$	
Redwood.........	5000	900	800,000	300	80	....	....	400	150	3300	900	680	$900(1 - l/60d)$	
Bald Cypress.....	4800	900	1,150,000	500	120	....	....	340	170	3900	1100	830	$1100(1 - l/60d)$	
Red cedar........	4200	800	860,000	....	....	....	....	470	230	2800	900	680	$900(1 - l/60d)$	
White oak........	5700	1100	1,150,000	840	210	270	110	920	450	3500	1300	980	$1300(1 - l/60d)$	12

NOTE.—The working unit stresses given in this table are intended for railroad bridges and trestles. For highway bridges and trestles the unit stresses may be increased 25 percent. For buildings and similar structures, in which the timber is protected from the weather and practically free from impact, the unit stresses may be increased 50 percent. To compute the deflection of a beam under long-continued loading instead of that when the load is first applied, only 50 percent of the corresponding modulus of elasticity given in the table is to be employed.

These unit stresses are for a green condition of timber and are to be used without increasing the live load stresses for impact.

* Partially air-dry.

l = length in inches.

d = least side in inches

given in the table. In order to minimize the results of sag, for beams that are fully loaded all the time, it is advisable to use one-half the values of modulus of elasticity given.

Working values are given for three conditions of exposure: (1) continuously dry, or use in interior or protected construction; (2) occasionally wet but quickly dried, as for such exterior construction as bridges, trestles, grandstands or bleachers, and exposed framework of open sheds; and (3) usually wet, as in the case of exposure to waves or tidewater, or in contact with earth.

These working stresses may be used without allowance for impact up to 100 percent.

In calculating the horizontal shear at one end of a beam, the concentrated loads between that end and a point distant three times the depth of the beam from it may be considered as acting at that point. In considering moving loads, as in highway or railway bridge stringers, in computing the shear at the end it is safe to ignore all wheel loads between that end and a point three times the depth of the beam or stringer from it, when the balance of the span is assumed loaded so as to give a maximum shear stress. Shear stresses for joint details may be taken as 100 percent greater than the values for horizontal shear given in the table.

For direct tension the same values as for extreme fiber stress in bending may be used. Grades of joists or beams may be used for members in direct tension, as in bottom chords of trusses.

Table 89*b* gives unit stresses that have been widely used in this country, having been the standard of the American Railway Engineering Association before the adoption of the above noted stresses.

ART. 90. WORKING UNIT STRESSES FOR COLUMNS

The associations mentioned in the first paragraph of Art. 89 recommend working stresses for columns, whose ratio of length to least dimension does not exceed 10, as given in Table 90*a*. For bearing on the ends of the fibers in joint details, these values may be increased one-third.

For columns of intermediate length, they recommend the formula

$$\frac{P}{A} = S\left[1 - \frac{1}{3}\left(\frac{l}{Kd}\right)^4\right] \qquad (1)$$

where P = total load in pounds;
A = required cross-sectional area in square inches;
S = safe stress in pounds per square inch in compression parallel to the grain for short columns;
l = unsupported length in inches;
d = least dimension in inches;
E = modulus of elasticity in pounds per square inch;
K = a coefficient depending on the species of timber.

This formula is to be used for values of $\frac{l}{d}$ between 10 and that value of $\frac{l}{d}$ which makes the right side of equation (1) equal to $\frac{2}{3}S$. For higher values of $\frac{l}{d}$, the Euler formula, with a factor of safety of 3, is recommended,

$$\frac{P}{A} = \frac{\pi^2}{36}\frac{E}{\left(\frac{l}{d}\right)^2} = 0.274\frac{E}{\left(\frac{l}{d}\right)^2} \tag{2}$$

The maximum slenderness ratio permitted is $\frac{l}{d} = 50$

Equations (1) and (2) are to be tangent to each other at $\frac{P}{A} = \frac{2}{3}S$, hence $\frac{P}{A} = \frac{2}{3}S = 0.274\frac{E}{\left(\frac{l}{d}\right)^2}$

or $\frac{l}{d} = \frac{\pi}{2}\sqrt{\frac{E}{6S}}$ at the point of tangency of the two curves

and $\frac{P}{A} = \frac{2}{3}S = S\left[1 - \frac{1}{3}\left(\frac{l}{Kd}\right)^4\right]$

or $K = \frac{l}{d}$

and $K = \frac{\pi}{2}\sqrt{\frac{E}{6S}}$

Hence K equals the value of $\frac{l}{d}$ at the point of tangency of the two curves. Table 90*a* gives values of K for a number of timbers used under different conditions.

TABLE 90*a*.—UNIT STRESSES AND VALUES OF *K* FOR COLUMN DESIGN

	Continuously Dry		Occasionally Wet		Usually Wet		Modulus of Elasticity
	Working Stress Lb. per Sq. In.	Value of *K*	Working Stress Lb. per Sq. In.	Value of *K*	Working Stress Lb. per Sq. In.	Value of *K*	
Cedar, western red, select	700	24.2	700	24.2	650	25.1	1,000,000
Common	560	27.1	560	27.1	520	28.1	1,000,000
Douglas fir, Coast region, select	1175	23.7	1065	24.9	905	27.0	1,600,000
Dense select	1285	22.6	1165	23.8	990	25.8	1,600,000
Coast region, common	880	27.3	800	28.6	680	31.1	1,600,000
Dense common	1025	24.9	935	26.1	795	28.3	1,600,000
Rocky Mountain region, select	800	24.8	800	24.8	700	26.5	1,200,000
Common	640	27.8	640	27.8	560	29.7	1,200,000
Hemlock, West coast, select	900	25.3	900	25.3	800	26.8	1,400,000
Common	720	28.3	720	28.3	640	30.0	1,400,000
Larch, western, select	1100	22.0	1000	23.1	800	25.8	1,300,000
Common	880	24.6	800	25.8	640	28.8	1,300,000
Pine, southern, dense select	1285	22.6	1165	23.8	990	25.8	1,600,000
Common	880	27.3	800	28.6	680	31.1	1,600,000
Dense common	1025	24.9	935	26.1	795	28.3	1,600,000
Redwood, select	1000	22.2	900	23.4	750	25.6	1,200,000
Common	800	24.8	720	26.1	600	28.6	1,200,000
Spruce, red, white, Sitka, select	800	24.8	750	25.6	650	27.5	1,200,000
Common	640	27.8	600	28.7	520	30.8	1,200,000

TABLE 90*b*.—VALUES OF $\left[1 - \frac{1}{3}\left(\frac{l}{Kd}\right)^4\right]$ IN FORMULA

$$\frac{P}{A} = S\left[1 - \frac{1}{3}\left(\frac{l}{Kd}\right)^4\right]$$

	RATIO OF LENGTH TO LEAST DIMENSION, $\frac{l}{d}$, IN RECTANGULAR TIMBERS																			
K	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
22	.97	.96	.95	.93	.91	.88	.85	.81	.77	.72	.67									
23	.98	.97	.95	.94	.92	.90	.87	.84	.81	.77	.72	.67	.	.	.	.	.	.	.	.
24	.98	.97	.96	.95	.93	.92	.89	.87	.84	.80	.76	.72	.67	.	.	.	.	.	.	.
25	.98	.98	.97	.96	.94	.93	.91	.89	.86	.83	.80	.76	.72	.67						
26	.99	.98	.97	.96	.95	.93	.92	.91	.89	.86	.83	.80	.76	.72	.67					
27	.99	.98	.98	.97	.96	.95	.93	.92	.90	.88	.85	.82	.79	.74	.71	.67				
28	.99	.98	.98	.97	.96	.95	.94	.93	.91	.89	.87	.85	.82	.79	.75	.71	.67			
29	.99	.99	.98	.98	.97	.96	.95	.94	.92	.91	.89	.87	.84	.82	.79	.75	.71	.67		
30	.99	.99	.98	.98	.97	.97	.96	.95	.94	.92	.90	.88	.86	.84	.81	.78	.75	.71	.67	
31	.99	.99	.99	.98	.98	.97	.96	.95	.94	.93	.92	.90	.88	.86	.84	.81	.78	.75	.71	.67

NOTE: — This table can also be used for columns not rectangular, the $\frac{l}{d}$ being equivalent to $0.289 \frac{l}{r}$, where r is the radius of gyration of the section.

Table 89*b* gives a formula for column designs which has been widely used, having been the standard of the American Railway Engineering Association before the adoption of the above formulas.

Table 90*b* gives values for the expression $\left[1 - \frac{1}{3}\left(\frac{l}{Kd}\right)^4\right]$ of Equation (1) which materially helps in design work.

ART. 91. WORKING COMPRESSIVE STRESSES ON SURFACES INCLINED TO THE FIBERS

A considerable amount of experimental work has been done to determine the bearing strength of timber on surfaces oblique to the grain but the results have been somewhat contradictory. Most of the formulas that have been advanced express the allowable working stress on surfaces inclined to the fibers in terms of the allowable stresses on the sides and ends of the fibers and of the angle made by the surface with the grain of the timber.

The Hankinson formula, developed from tests made by the War Department and published by the Chief of Air Service in 1921 in a bulletin entitled "Investigation of Crushing Strength of Spruce at Varying Angles of Grain," conforms closely to the results of many tests and is as follows:

$$s = \frac{p}{\frac{p}{q}\cos^2\theta + \sin^2\theta} \text{ or } s = Kp,$$

$$\text{where } K = \frac{1}{\frac{p}{q}\cos^2\theta + \sin^2\theta}$$

s = unit allowable compression on the inclined plane;
p = unit allowable compression on the ends of the fibers;
q = unit allowable compression on the sides of the fibers (perpendicular to the grain);
θ = the angle made by the plane with the direction of the fibers.

Figure 91*a* will expedite the use of this formula in computation work. For example, to get the allowable bearing on dense-select southern pine protected from the weather on a plane at an angle of 30 deg. with the fibers, using the American Railway Engineering Association units as given in Arts. 89 and 90, we find that $p/q = 1713/380 = 4.5$. Entering the diagram along the bottom at $\theta = 30$ deg. and going up vertically to $p/q = 4.5$, we

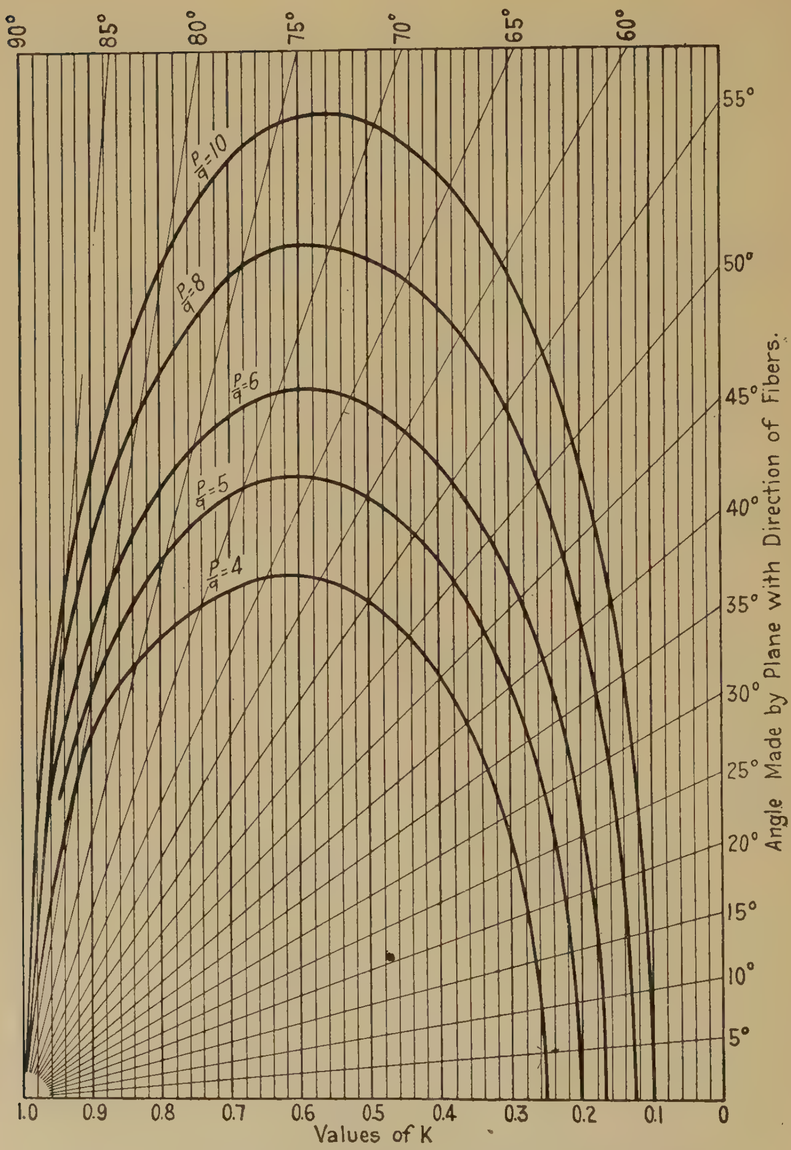

FIG. 91*a*.—Coefficients for Allowable Stresses on Inclined Planes.

find that K is 0.28; hence $s = 0.28 \times 1713 = 480$ lb. per sq. in. If these curves are drawn on tracing cloth and the base line is placed on the drawing parallel to the fibers the value of K for any cut in the timber can be read directly without finding the angle θ.

Another formula that has been widely quoted is the Howe formula:

$$s = q + (p - q)\ (\theta/90)\ 5/2$$

This formula agrees closely with the Hankinson formula, the latter giving somewhat higher values for angles near 90 deg.

The Jacoby formula, which appeared in the first edition of this text, has been widely used in practice and with apparently satisfactory results. It gives values considerably higher than experiments appear to justify. It is as follows:

$$s = p \sin^2 \theta + q \cos^2 \theta$$

ART. 92. REFERENCE HANDBOOKS

Instead of reprinting a number of tables for the use of students and designers in heavy framing, references are given to the principal handbooks which are to be found in the offices of engineers, architects, builders, etc., and in the libraries of technical colleges. The designer should have some of these constantly at hand for use, and should be thoroughly familiar with their contents.

Arthur, William. New Building Estimator. Scientific Book Corp., New York. 4¾ by 6¾ in.

Gillette, Halbert, P. Handbook of Cost Data. Gillette Pub. Co., New York. 4½ by 7 in.

Hearne, H. C. Table of Stresses in Roof Trusses. McGraw-Hill Book Co., New York.

Hool and Johnson. Handbook of Building Construction. McGraw-Hill Book Co., New York. 6 by 9 in.

Hool and Kinne. Steel and Timber Structures. McGraw-Hill Book Co., New York. 6 by 9 in.

Inskip, G. D. Tables of Five-place Squares and Logarithms. New York. 5½ by 7 in.

International Correspondence School. Building Trades Pocket-book. Scranton, Pa. 3½ by 5¼ in.

Kent, William. Mechanical Engineers' Pocket-book. John Wiley & Sons, New York. 4¼ by 6¾ in.

Kidder, F. E. Architects' and Builders' Pocket-book. John Wiley & Sons, New York. 4¼ by 6¾ in.

Merriman, M. American Civil Engineers' Handbook. John Wiley & Sons, New York. $4\frac{1}{2}$ by 7 in.

Richey, H. G. Guide and Assistant for Carpenters and Mechanics. Comstock, New York, $4\frac{1}{4}$ by $6\frac{3}{4}$ in.

Smoley, Constantine. Tables, Containing Parallel Tables of Logarithms and Squares. C. K. Smoley & Sons, Chicago, $4\frac{3}{4}$ by 7 in.

Southern Pine Manual. Southern Pine Association, New Orleans. $4\frac{1}{4}$ by $6\frac{1}{2}$ in.

Structural Timber Handbook. West Coast Lumbermen's Association, Seattle. 5 by $7\frac{3}{4}$ in.

Trautwine, John C. Revised by John C. Trautwine, Jr., and John C. Trautwine, 3d. Civil Engineer's Pocket-book. Trautwine Co., Philadelphia. $4\frac{1}{4}$ by $6\frac{3}{4}$ in.

Voss and Varney. Architectural Construction, Vol. 2, Book 1. John Wiley & Sons, New York. 9 by $11\frac{1}{2}$ in.

Winslow, Benjamin E. Tables. Diagrams for Calculating the Strength of Wood, Steel, and Cast-iron Beams and Columns. McGraw-Hill Book Co., New York. 12 by 9 in.

The following handbooks are issued by manufacturers of structural materials:

Carnegie Steel Co. (Pittsburgh, Pa.). Pocket Companion Containing Useful Information and Tables Appertaining to the Use of Steel.

Carnegie Steel Co. Carnegie Beam Section Profiles, Properties and Safe Loads for New Series of Structural Steel Beam and Column Sections.

Bethlehem Steel Co. (Bethlehem, Pa.). Bethlehem Structural Shapes.

American Institute of Steel Construction. Steel Construction.

Jones & Laughlin Steel Co. (Pittsburgh, Pa.). List and Diagram of Shapes.

Phoenix Iron Co. (Phoenixville, Pa.). Handbook of Useful Information, Tables, Data, and Formulas Appertaining to the Use of Steel.

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